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A Comparison of Oxidation Ditch Plants to Competing Processes for Secondary and Advanced Treatment of Municipal Wastes

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A COMPARISON OF OXIDATION DITCH PLANTS TO COMPETING PROCESSES FOR SECONDARY AND ADVANCED TREATMENT OF MUNICIPAL WASTES

Municipal Environmental Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

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This report includes information relating to oxidation ditch plant plant equipment, design and application, operational problems and advantages, operation and maintenance requirements, construction costs, and nitrification and nitrogen removal applications. Much of the information is based on visits to and analysis of data from actual operating installations. In addition, the oxidation ditch plant characteristics are compared to those of competing biological treatment processes. Nitrification and nitrogen removal capabilities of the oxidation ditch process are also compared to various biological and physical-chemical processes.

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This report includes information relating to oxidation ditch plant equipment, design and application, operational problems and advantages, operation and maintenance requirements, construction costs, and nitrification and nitrogen removal applications. Much of the information is based on visits to and analysis of data from actual operating installations. In addition, the oxidation ditch plant characteristics are compared to those of competing biological treatment processes. Nitrification and nitrogen removal capabilities of the oxidation ditch process are also compared to various biological and physical-chemical processes.

Francis T. Mayo, Director
Municipal Environmental Research
Laboratory

EXECUTIVE SUMMARY

INTRODUCTION

The purpose of this study is to present state-of-the-art information regarding oxidation ditch plant design, costs, operation, performance, and reliability. This information is used to compare oxidation ditch plants to other competing biological processes. Design criteria and operational conditions necessary to maximize single stage nitrification and total nitrogen removals in oxidation ditch plants is also presented. Costs of competing nitrification and nitrogen removal processes are compared to those developed for oxidation ditch plants.

DESIGN FEATURES OF OXIDATION DITCH PLANTS

An oxidation ditch plant is typically an extended aeration type of activated sludge process that uses a continuously recirculating closed loop channel or channels as an aeration basin. The aeration basin is normally sized for a 24-hour hydraulic retention time, but may be designed for any other detention time. Mechanical aerators are commonly used for mixing, oxygen supply, and for circulation of mixed liquor. Generally, these are horizontal brush, cage, or disc-type aerators designed specifically for oxidation ditch plants. Secondary clarifiers similar to those used in other activated sludge processes are normally provided. Primary clarification is not usually included in oxidation ditch plant design.

The typical oxidation ditch aeration basin is a single channel or multiple interconnected concentric channels. An oxidation ditch plant normally consists of one or more basins of either type operated in parallel depending on the flow and operation mode required. Channel geometry can vary to include many possible configurations, however, the oval configuration is the most common. The multiple concentric channel basin can have any number of interconnected channels with three to five being typical and provides some process flexibility since it can be changed to other activated sludge modes with minor modification. Typically, the outer channel is used for aerobic digestion of the waste activated sludge. Shallow channels are typically four to six feet deep with 45 degree sloping side walls. Deep channels have vertical side walls and are normally 10-12 feet deep. The channels are usually lined to prevent erosion and leakage. Ditch lining can be reinforced concrete, gunite, asphalt, or thin membranes. Typically, shallow channels with sloped side walls are constructed of concrete poured against earth backing with welded wire mesh reinforcing. Deep vertical wall channels require reinforced concrete walls.

Several manufacturers supply oxidation ditch brush or disc type mechanical aerators. These units may be either fixed or floating. The aerators normally span the channel width and may be installed in one or more locations around the channel. The aerators must supply the required oxygen to the channel and impart a sufficient velocity in the channel (>1.0 FPS) to keep the channel contents in suspension. Oxygen transfer capabilities of an aerator will vary depending on the particular design, rotational speed and submergence. Most units operate in the range of 60 RPM to 110 RPM with a submergence of 2 to 12 inches and produce oxygen transfer rates of from 3 to 5 lbs of oxygen per hour. The number of aerators provided depends on the size, configuration and oxygen requirements of the plant. A minimum of two aerators should be installed so that at least partial aeration can be provided when problems occur.

Generally, final clarifier design is consistent with other activated sludge processes. A surface overflow rate of 400 to 500 gpd/sq ft is recommended for average daily flows and 1000 to 1200 gpd/sq ft at peak flows. Many plants are constructed with 8 foot deep clarifiers, but depths of 10 to 14 feet provide greater process reliability.

COMPARATIVE PERFORMANCE & RELIABILITY

Performance data for the 29 oxidation ditch plants studied are summarized as follows:

SUMMARY PERFORMANCE OF 29 OXIDATION DITCH PLANTS

	Effluent, mg/l			Removal		
	Winter	Summer	Average annual	Winter	Summer	Average annual
BOD_5						
High plant	55	34	41	87	86	87
Average	15.2	1.2	12.3	92	94	93
Low plant	1.9	1.0	1.5	99	99	99
Suspended solids						
High plant	26.6	19.4	22.4	81	82	82
Average	13.6	9.3	10.5	93	94	94
Low plant	3.1	1.9	2.4	98	98	98

The performance of competing biological treatment processes was also evaluated and the data are summarized as follows:

PERFORMANCE - COMPETING BIOLOGICAL PROCESSES

	Effluent, mg/l		Removal, %	
	TSS	BOD_5	TSS	BOD_5
Activated sludge (1.0 mgd)	31	26	81	84
Activated sludge (Package Plants)	28	18	-	-
Trickling filters	26	42	82	79
Rotating biological contactor	23	25	79	78

Analysis of data from 12 operating oxidation ditch plants showed the reliability for meeting various BOD_5 and TSS effluent standards as follows:

RELIABILITY - OXIDATION DITCH PLANTS

	% of time effluent concentration (mg/l) less than					
	10 mg/l		20 mg/l		30 mg/l	
	TSS	BOD_5	TSS	BOD_5	TSS	BOD_5
Best plant	99%	99%	99%	99%	99%	99%
Average all plants	65%	65%	85%	90%	94%	96%
Worst plant	25%	20%	55%	55%	80%	72%

Of the plants analyzed, the effluent BOD_5 and TSS seldom exceeded a maximum of 60 mg/l.

The reliability of competing biological treatment processes was evaluated on the same basis and is summarized as follows:

AVERAGE RELIABILITY - COMPETING BIOLOGICAL PROCESSES

	% of time effluent concentration (mg/l) less than					
	10 mg/l		20 mg/l		30 mg/l	
	TSS	BOD_5	TSS	BOD_5	TSS	BOD_5
Activated sludge (1.0 mgd)	40	25	75	70	90	85
Activated sludge (Package Plants)	15%	39%	35%	65%	50%	80%
Trickling filters	-	2%	-	3%	-	15%
Rotating biological contactor	22%	30%	45%	60%	70%	90%

An oxidation ditch plant is capable of 95% to 99% nitrification without design modifications. This high degree of nitrification even at wastewater temperatures approaching 0°C is possible due to the 24-hour hydraulic retention time in the channel(s) and the capability of operating at a high solids retention time (SRT) of 10 to greater than 50 days.

Nitrogen removal by single-stage biological nitrification-denitrification has also been achieved at properly designed and well operated oxidation ditch plants. Nitrogen removal is achieved by producing both aerobic and anoxic zones within the same channel. These zones are created by controlling the aerator oxygen transfer rate so that mixed liquor dissolved oxygen is depleted within a portion of the aeration channel.

The carbon source for the anoxic zone (denitrification) is provided by feeding the raw sewage into the channel upstream of the anoxic zone. With careful operation, 80% nitrogen removal has been achieved in a single channel oxidation ditch plant.

OXIDATION DITCH PLANT OPERATION

Oxidation ditch plants can be operated by average personnel to produce above average performance results. Assuming no mechanical malfunction, oxidation ditch plants are capable of performing well for several days at a time with minimal operator attention.

Waste activated sludge handling requirements depend on the plant design and operation. Plants operated with a 24-hour channel hydraulic retention time at 20 to 30 day SRT produce a biologically stable waste sludge that can be handled without causing significant odor problems. Plants operated with 6 to 8 hour channel hydraulic retention time and less than 10 day SRT require additional sludge treatment, typically aerobic digestion. Some problems have been noted in dewatering activated sludge directly on sand beds from plants operated at a 24-hour hydraulic retention time. These sludges tend to dewater very slowly requiring significantly increased sludge drying bed area. In areas where wet and cold weather are common, sludge drying bed area should be even larger. Some plants are operated indefinitely without formal sludge wasting allowing solids to build up in the aeration channel. This type of operation is considered marginal because the plant is prone to periodic clarifier upsets resulting in high final effluent solids.

Aerators and drives have typically required major corrective maintenance every 2-5 years for the following type problems.

1. Bearing and seal failure due to improper selection, constant water splashing on the bearings and seals, and settlement of the aerator support structure causing misalignment.

2. Loss of aerator elements due to corrosion.

3. Aerator torque tube failure or excessive deflection of very long units.

COMPARATIVE COSTS

Costs associated with oxidation ditch plants were determined. Construction costs for 44 plants were escalated by EPA Treatment Plant Index 262.3 for the third quarter of 1976. These costs include all facilities except land, engineering, legal and financing during construction. Annual

O & M costs include labor, utilities, chemicals, maintenance materials, and miscellaneous. These costs are as follows:

OXIDATION DITCH PLANT CONSTRUCTION AND ANNUAL O & M COSTS

	$1000, 1976		
Plant capacity, mgd	0.1	1.0	10.0
Construction	195	600	3350
O & M normal	22.1	62.4	446.6
O & M nitrification	22.6	63.1	467.5
O & M N-removal	28.1	67.4	453.7

There are generally no increased construction costs for nitrification or nitrogen removal.

Construction and O & M costs were developed for competing activated sludge processes. These costs were developed on the same basis as the oxidation ditch costs. The construction and annual O & M costs are as follows:

CONSTRUCTION COSTS - COMPETING ACTIVATED SLUDGE PROCESSES

	$1000, 1976			
Capacity, mgd	0.5	1.0	5.0	10.0
Extended aeration (package plants)	390	-	-	-
Contact stabilization (package plants)	320	475	-	-
Conventional activated sludge	-	1045	2645	4138

ANNUAL O & M COSTS - COMPETING ACTIVATED SLUDGE PROCESSES

	$1000, 1976			
Capacity, mgd	0.5	1.0	5.0	10.0
Extended aeration (package plants)	64.4	-	-	-
Contact stabilization (package plants)	57.3	93.9	-	-
Conventional activated sludge	-	80.9	187.7	308.1

Construction and annual O & M costs were developed for competing biological nitrification. These are summarized as follows for 20 mg/l influent NH_4-N.

CONSTRUCTION COSTS FOR COMPETING BIOLOGICAL NITRIFICATION PROCESSES

Capacity, mgd	$1000, 1976 1	 5	 10
Activated sludge, 1 stage (20 mg/l NH_4-N)	1,210	3,203	5,107
Activated sludge, 2 stage (20 mg/l NH_4-N)	1,448	3,830	6,031

ANNUAL O & M COSTS FOR COMPETING BIOLOGICAL NITRIFICATION PROCESSES

Capacity, mgd	$1000, 1976 1	 5	 10
Activated sludge, 1 stage (20 mg/l NH_4-N)	89.4	219.3	375.5
Activated sludge, 2 stage (20 mg/l NH_4-N)	102.9	245.4	416.9

Construction and annual O & M costs were developed for competing biological denitrification processes. These are summarized as follows for 20 mg/l influent NH_4-N.

CONSTRUCTION COSTS FOR COMPETING BIOLOGICAL DENITRIFICATION PROCESSES

Capacity, mgd	$1000, 1976 1	 5	 10
Mixed reactor denitrification (20 mg/l NH_4-N)	539	1,357	2,291
Fixed film denitrification (20 mg/l NH_4-N)	636	1,192	2,298

ANNUAL O & M COSTS FOR COMPETING BIOLOGICAL DENITRIFICATION PROCESSES

Capacity, mgd	$1000, 1976 1	 5	 10
Mixed reactor denitrification (20 mg/l NH_4-N)	54.1	140.6	244.6
Fixed film denitrification (20 mg/l NH_4-N)	51.3	115.3	195.0

Construction and annual O & M costs were also developed for selected physical-chemical nitrogen removal processes. These are summarized as follows for 20 mg/l influent NH_4-N.

CONSTRUCTION COSTS FOR COMPETING PHYSICAL-CHEMICAL NITROGEN REMOVAL PROCESSES

	$1000, 1976		
Capacity, mgd	1	5	10
Breakpoint chlorination (20 mg/l NH_4-N)	114.8	377.1	696.7
Selective ion exchange (20 mg/l NH_4-N)	442.6	1,557.4	2,704.9
Ammonia stripping (20 mg/l NH_4-N)	245.9	1,065.6	1,967.2

ANNUAL O & M COSTS FOR COMPETING PHYSICAL-CHEMICAL NITROGEN REMOVAL PROCESSES

	$1000, 1976		
Capacity, mgd	1	5	10
Breakpoint chlorination (20 mg/l NH_4-N)	100.0	280.0	380.0
Selective ion exchange (20 mg/l NH_4-N)	47.0	150.0	250.0
Ammonia stripping (20 mg/l NH_4-N)	18.0	57.0	170.0

Construction costs for competing extended aeration and contact stabilization plants were less than for oxidation ditch plants in the flow range of 0.01 mgd to 2 mgd. Oxidation ditch plant construction costs were less than for conventional activated sludge plants within the range of 0.01 to 10 mgd. Operation and maintenance costs for oxidation ditch plants were less than for the competing processes in 0.1 mgd to 2 mgd range and the total annual costs for oxidation ditch plants were less than for all other competing processes in the range of 0.1 to 10 mgd. Within the flow range of 0.01 to 0.1 mgd the total annual costs for extended aeration package plants was less than for oxidation ditch plants. The total annual costs for all competing denitrification processes were higher than for oxidation ditch plants.

CONCLUSIONS

The results of this study show that oxidation ditch plants are capable of consistently achieving high levels of BOD_5 and TSS removals with minimum operation. High levels of nitrification (95%-99%) are possible with proper operation. Nitrogen removals as high as 80% can be achieved in a single channel plant with careful operation of the aeration equipment to produce aerobic and anoxic zones within the channel. Increased operator attention is required to produce the high levels of nitrogen removal.

Cost data developed in this report show that oxidation ditch plants are competitive with other biological processes. Total annual costs for oxidation ditch plants were less than for all competing biological

processes within the flow range of 0.1 to 10 mgd. Oxidation ditch plants were also shown to have lower total annual costs than competing biological and physical-chemical nitrogen removal processes.

This report was submitted in partial fulfillment of Contract No. 68-02-2186 by Culp/Wesner/Culp - Clean Water Consultants under sponsorship of the U.S. Environmental Protection Agency. This report covers the period of July, 1976 to December, 1976, and work was completed as of August, 1977.

CONTENTS

Foreword iii
Executive Summary. iv
Figures. xv
Tables xviii

1. Introduction 1
    Study objectives. 1
    Study procedure 1
    History 1
    Current trends in the United States 2
2. Designs and Applications 6
    Pretreatment. 6
    Oxidation ditch plant and arrangements and designs. 9
    Sludge handling 14
    Oxidation ditch unique equipment suppliers. . 15
    Evaluation of designs 29
3. Performance and Reliability. 47
    General 47
    Performance 48
    Reliability 48
    Oxygen uptake rate. 57
    Mixed liquor characteristics. 57
    Solids production 59
    Effect of oxidation ditch configurations on process performance 59
4. Nitrification and Nitrogen Removal 61
    General 61
    Nitrification 61
    Design parameters for nitrification 68
    Operational parameters for nitrification. . . 70
    Design parameters for nitrogen removal. . . . 72
    Operational parameters for nitrogen removal . 75
    Summary 76
5. Operation. 78
    Oxidation ditch plant problems. 78
    Operation and maintenance requirements. . . . 82
    Nitrogen removal. 86
6. Construction 87
    Construction cost 87
    Nitrification construction costs. 87
    Nitrogen removal construction costs 87

Preceding page blank

Equipment pricing 91
Ditch configuration 91
Plant area. 91
Cold climate. 96
Other factors 96
7. Competing Processes. 97
General 97
Capability and reliability of competing biological processes 97
Performance summary 115
Costs of competing biological processes 115
Biological nitrification. 129
Biological denitrification. 136
Physical chemical nitrogen removal. 136
Summary and comparison. 145
8. Discussion and Evaluation. 158
Process and design. 158
Process equipment 158
Performance 159
Construction. 159
Operation and maintenance 160
Sludge handling 162
Nitrification and nitrogen removal. 163
Effective application 165

References 166
Bibliography 169
List of Metric Conversions 171

FIGURES

Number Page

1 Municipal oxidation ditch plant installations in the United States . . . 5

2 Single channel oxidation ditch plant and typical channel configuration . . . 7

3 Multiple concentric channel oxidation ditch plant . . . 8

4 Typical brush type aerator . . . 17

5 Typical disc type aerator . . . 18

6 Lakeside Cage and Mini-Magna aerator characteristics . . . 20

7 Lakeside Magna-Rotor characteristics . . . 21

8 Passavant Series 5200 aerator performance, 90 rpm . . . 22

9 Passavant Series 5300 aerator performance, 70 rpm . . . 23

10 Envirex 4'-6" aeration disc characteristics . . . 24

11 Cherne "OTA aerator" aerator configuration . . . 26

12 Cherne "OTA aerator" characteristics . . . 27

13 Envirotec "Carrousel" schematic . . . 28

14 Horizontal aerator efficiency . . . 31

15 Carbonaceous BOD oxygen requirements . . . 35

16 Effect of organic variations . . . 37

17 Effect of hydraulic variations . . . 41

18 Biological process mixed liquor relationships and solids production . . . 45

19 BOD_5 and TSS removal performance . . . 52

Number Page

20 Oxidation ditch plant BOD_5 reliability 53

21 Oxidation ditch plant suspended solids reliability 54

22 Oxidation ditch plant COD reliability 55

23 Oxidation ditch plant total nitrogen reliability 56

24 Concentric channel nitrogen removal flow diagram (Drews)(17)(18) 65

25 Oxidation ditch plant average construction cost 89

26 Effluent quality, trickling filters 98

27 Trickling filter effluent quality, two Texas plants 100

28 Activated sludge effluent quality 104

29 Activated sludge effluent quality, Dallas, Texas nitrification pilot plant and El Lago, Texas 106

30 Filtered activated sludge plant BOD_5 quality based on four plants 108

31 Activated sludge package plant reliability, BOD_5 110

32 Activated sludge package plant reliability, suspended solids 111

33 RBC effluent quality, Gladstone, Michigan 113

34 RBC effluent quality monthly data with chemical coagulation, Gladstone, Michigan 114

35 Activated sludge process schematic 123

36 Biological treatment process construction cost, 1976 147

37 Biological treatment process operation and maintenance cost, 1976 148

38 Biological treatment process total annual cost, 1976 149

39 Incremental construction cost for biological nitrification, 1976 152

40 Incremental operation and maintenance cost for biological nitrification, 1976 153

Number Page

41 Incremental construction cost for biological and physical-chemical denitrification, 1976 154

42 Incremental operation and maintenance cost for biological and physical-chemical denitrification 1976 155

43 Incremental total annual cost for biological and physical-chemical denitrification, 1976 156

TABLES

Number — Page

1 Municipal Oxidation Ditch Plant Installations 3

2 Typical Single Channel Designs - Oxidation Ditch Plants 12

3 Typical Multiple Concentric Channel Design - Oxidation Ditch Plants 13

4 Comparative Aerator Characteristics 16

5 Source of Performance Data 49

6 Oxidation Ditch Plant Performance Summary. 50

7 Sources of Reliability Data and Plant Characteristics. 51

8 Oxidation Ditch Plant Dissolved Oxygen and Mixed Liquor Suspended Solids. 58

9 Typical Mixed Liquor Settleability 60

10 Ammonia and Total Nitrogen Performance Data. 62

11 Multi Channel Oxidation Ditch Nitrogen Removal Results (Drews)(17)(18). 67

12 Sources of Oxidation Ditch Plant O & M Information . . 84

13 O & M Requirements, Oxidation Ditch Plants, 1976 . . . 85

14 Sources of Oxidation Ditch Plant Construction Cost Information. 88

15 Oxidation Ditch Plant Construction Cost, 1976. 90

16 Incremental Construction Costs For Nitrogen Removal. . 92

17 Oxidation Ditch Plant Major Equipment Cost, 1976 . . . 93

18 Comparative Construction Cost Of Oxidation Ditch Configurations 94

Number Page

19 Approximate Oxidation Ditch Plant Area Requirements. 95

20 Trickling Filter Summary, Plant Visits 101

21 Trickling Filter Summary, EPA Region Data. 102

22 Activated Sludge Summary, EPA Region Data. 103

23 Summary of Competing Process Performance 116

24 Construction Cost of Contact Stabilization Plants, 1976 118

25 Operation and Maintenance Costs, Contact Stabilization Plants, 1976 119

26 Construction Cost of Extended Aeration Plants, 1976. . 120

27 Operation and Maintenance Costs, Extended Aeration Plants, 1976 121

28 Design Parameters, Activated Sludge 124

29 Unit Process Sizes, Activated Sludge 125

30 Construction Costs, Activated Sludge, 1976 126

31 Operation and Maintenance Costs, Activated Sludge, 1976 127

32 Design Parameters, Single Stage Activated Sludge Nitrification 130

33 Unit Process Sizes, Single Stage Activated Sludge Nitrification. 131

34 Construction Cost, Single Stage Activated Sludge Nitrification 132

35 Design Parameters, Two Stage Activated Sludge Nitrification 133

36 Unit Process Sizes, Two Stage Activated Sludge Nitrification. 134

37 Construction Cost, Two Stage Nitrification, 1976 . . . 135

38 Design Parameters, Mixed Reactor Denitrification . . . 137

Number Page

39 Unit Process Sizes, Mixed Reactor Denitrification. . . 138

40 Construction Costs, Mixed Reactor Denitrification, 1976 139

41 Operation and Maintenance Costs, Mixed Reactor Denitrification, 1976. 140

42 Construction Costs, Fixed Film Denitrification 141

43 Operation and Maintenance Costs, Fixed Film Denitrification, 1976. 142

44 Construction Costs, Physical-Chemical Nitrogen Removal, 1976. 143

45 Operation and Maintenance Costs, Physical-Chemical Nitrogen Removal, 1976 144

46 Operating Plant Operation and Maintenance Costs. . . . 150

47 Characteristics of Biological Treatment Processes (For Range of Sizes Considered in This Study). . . . 151

48 Characteristics of Nitrogen Removal Processes. 157

SECTION 1

INTRODUCTION

STUDY OBJECTIVES

There are two major objectives to this study of oxidation ditch plants. First, is to compare the cost, performance, and reliability of oxidation ditch plants with conventional activated sludge plants in the size range of 1-10 mgd* and contact stabilization, and extended aeration activated sludge package plants within the size range of 0.01 to 1 mgd. Various manufacturers' oxidation ditch equipment and design recommendations will be studied and compared.

The second major objective is to summarize the design criteria and operational conditions necessary to maximize single stage nitrification and total nitrogen removal in oxidation ditch plants and to compare the total costs to competing processes in the size range 0.01 to 10 mgd.

This study is intended to provide useful and state of the art information on oxidation ditch plants to a wide audience of designers responsible for planning, evaluating, selecting and designing municipal wastewater treatment plants.

STUDY PROCEDURE

The work reported in this study is based on manufacturers literature, published literature, information from consultants and others knowledgeable in the subject and information from operating installations. The information from operating installations was obtained by visiting approximately 20 operating plants by letter and telephone contacts to 20 others.

HISTORY

The oxidation ditch was developed during the 1950's at the Research Institute for Public Health Engineering (TNO) in the Netherlands as an easily operated and low-cost method of treating raw sewage emanating from small communities and industries.

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*English units are used uniformly in this report because many of the English measures are common in the sanitary field. Conversion factors are contained in the List of Metric Conversions.

The first oxidation ditch plant was reportedly placed in service in 1954 at Voorshoten, Holland. This plant was designed in accordance with the principles of Dr. Ir. A. Pasveer of TNO for a population equivalent of 360 persons. This plant was an intermittent flow type in which the ditch also served as the final clarifier.

Since that time, oxidation ditch plants of many different configurations and designs have been placed in operation throughout the world.

The first major USA installations were made in the early 1960's. The equipment for these early installations was furnished by Lakeside Equipment Corp. Since that time the number of installations has increased and the following companies now manufacture or market oxidation ditch plant equipment in this country.

Lakeside Equipment Corporation

Passavant Corporation

Envirex, Incorporated

Walker Process

Cherne Industrial, Inc.

Envirotech Corporation (Carrousel License)

In addition, several firms market equipment for oxidation ditch plants used specifically in treatment of animal wastes including Fairfield Engineering and Manufacturing Company and Thrive Centers, Inc.

As of late 1975, manufacturers literature indicated the following number of municipal oxidation ditch plant installations in the United States.

Lakeside	467
Passavant	56
Envirex	20
Cherne	1

There are many "rotor" type aerators installed in other aeration applications such as aerated lagoons, extended aeration reactors, and aerobic digesters. These applications are not considered as oxidation ditch plants.

CURRENT TRENDS IN THE UNITED STATES

The approximate distribution of oxidation ditch plant installations by State and Canadian Province is shown in Table 1. The approximate

TABLE 1. MUNICIPAL OXIDATION DITCH PLANT INSTALLATIONS

STATES	LAKESIDE	PASSAVANT	ENVIREX	CHERNE	TOTAL
ALABAMA		1			1
ALASKA	3				3
ARIZONA					0
ARKANSAS	1				1
CALIFORNIA	16	1			17
COLORADO	13				13
CONNECTICUT					0
DELAWARE					0
FLORIDA	2	1			3
GEORGIA	21	14	3		38
HAWAII					0
IDAHO	4				4
ILLINOIS	4		2		6
INDIANA	12	1			13
IOWA	4	3			7
KANSAS	1				1
KENTUCKY	1				1
LOUISIANA	13	3	1		17
MAINE	23				23
MARYLAND		1			1
MASSACHUSETTS	3				3
MICHIGAN					0
MINNESOTA	7			1	8
MISSISSIPPI	2				2
MISSOURI	17	1			18
MONTANA					0
NEBRASKA	5	1			6
NEVADA	1				1
NEW HAMPSHIRE	8	1			9
NEW JERSEY	4	1			5
NEW MEXICO	5	2			7
NEW YORK	7	2			9
NORTH CAROLINA	7		1		8
NORTH DAKOTA					0
OHIO	21	3	1		25
OKLAHOMA	2				2
OREGON	6				6
PENNSYLVANIA	7	1			8
RHODE ISLAND					0
SOUTH CAROLINA	20	2	8		30
SOUTH DAKOTA	1				1
TENNESSEE	3				3
TEXAS	165	16	17		198
UTAH					0
VERMONT	10				10
VIRGINIA					0
WASHINGTON	29	1			30
WEST VIRGINIA	14				14
WISCONSIN			1		1
WYOMING	5				5
TOTAL USA	467	56	34	1	558

PROVINCES	
ALBERTA	10
BRITISH COLUMBIA	24
MANITOBA	1
NEW BRUNSWICH	12
NEWFOUNDLAND	3
NOVA SCOTIA	22
ONTARIO	17
PRINCE EDWARD ISLAND	0
QUEBEC	0
SASKATCHEWAN	1
TOTAL CANADA	90

number of United States installations by year are shown in Figure 1. All of the installation data were developed from manufacturers published installation lists covering the period through 1975.

All information indicates that the current trend is toward increased numbers of oxidation ditch plants especially in the size range up to 1.5 mgd. Only a few larger plants are presently in operation. There are only a few states with no actual or planned oxidation ditch plant installations.

Increased installation of oxidation ditch plants is related to some or all of the following considerations. These factors are discussed in detail later in the study.

1. Construction cost equal to or less than competitive treatment processes.

2. Plants require a minimum of mechanical equipment.

3. Plants appear to perform reasonably well even with minimum operator attention, primarily due to conservative design.

4. Waste sludge is relatively nuisance free and is readily disposed of at most plants.

5. Plants generally do not generate odors even under poor operating conditions.

Discussions with plant operators, public works officials, consulting engineers, and others who have had direct experience with oxidation ditch plants indicate a high level of satisfaction with and acceptance of oxidation ditch plants. There are a few exceptions and there are a few oxidation ditch plants that have been removed from service for various operational reasons, but these are a small minority.

There are no apparent factors that would cause the rate of application of oxidation ditch plants in the United States to decrease in the near future. There will be cyclic variations from year to year, but the trend should be increasing. Major changes in the EPA Construction Grants Program or regulatory requirements could affect these trends.

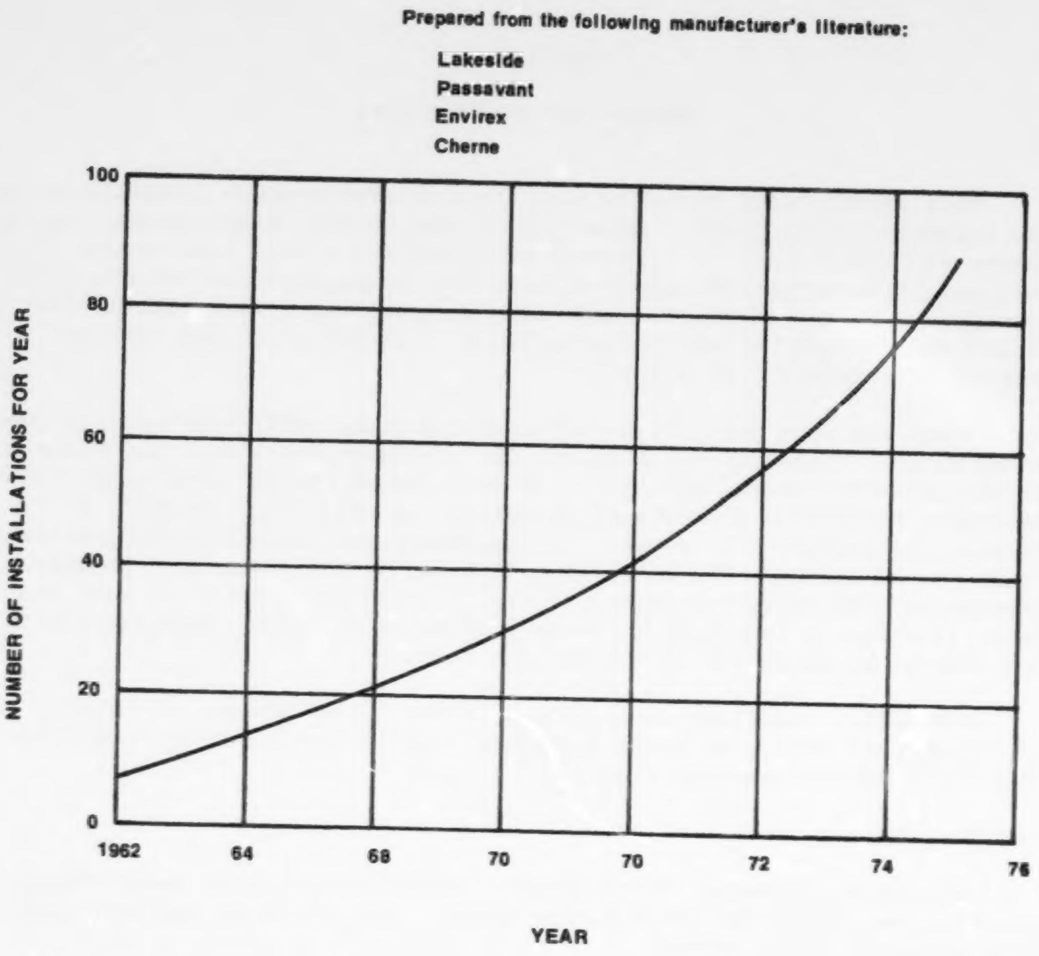


Figure 1. Municipal oxidation ditch plant installations in the United States.

SECTION 2

DESIGNS AND APPLICATIONS

This section will review various process arrangements historically used for oxidation ditch plants, arrangements used in the United States, design parameters recommended by equipment suppliers and a technical review of the designs. Present preliminary treatment and sludge handling and digestion practices will be discussed. Each manufacturers' equipment will be described and differences will be determined. Nitrification and nitrogen removal is considered in Section 4.

There are many possible oxidation ditch plant configurations and operating modes. Oxidation ditch plants can be adapted to almost any variation of the activated sludge process. Oxidation ditch plants are constructed to provide a continuous ring shaped circuit(s) or channel(s) in which an aerator (or aerators) is mounted to provide oxygen transfer and circulation of the mixed liquor. The mixed liquor passes by the aerator at regular intervals. The dissolved oxygen profile around the channel can vary significantly which is important to certain operational modes, especially nitrogen removal as discussed in a following section.

Generally, oxidation ditch plants in the United States are of the single channel design as shown in Figure 2 or of the multiple concentric channel design as shown in Figure 3.

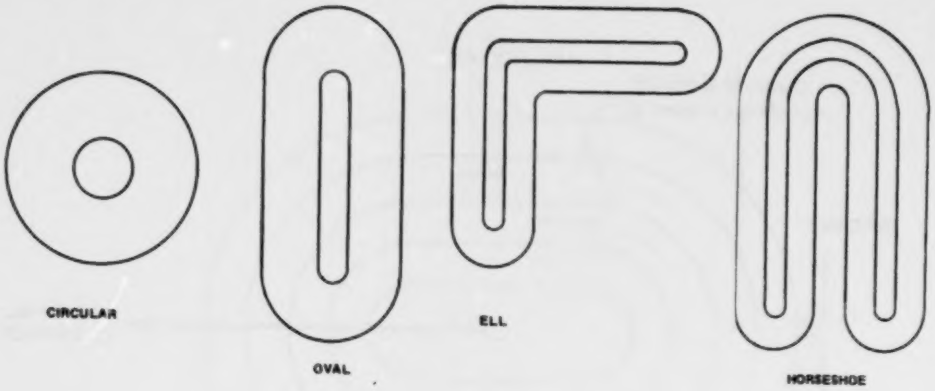
PRETREATMENT

Generally, oxidation ditch plants contain some form of pretreatment prior to the raw sewage entering the ditch. The following has been used with oxidation ditch plants.

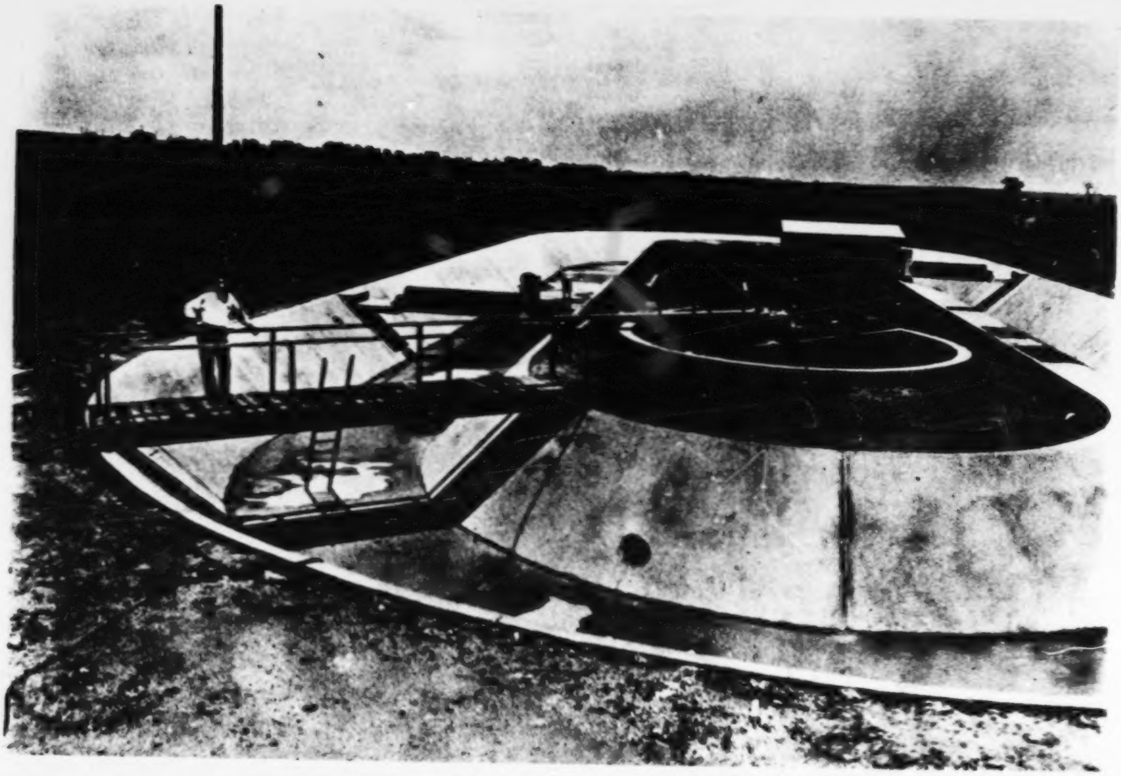
1. Coarse Screening
    a. Manually cleaned bar rack (typical opening size ½ to 1 in)
    b. Barminutor w/bypass manual bar rack

2. Comminutor with bypass manual bar rack
    a. Continuous
    b. Oscillating

3. Grit Removal
    a. Manually cleaned grit chamber
        (1) Non aerated
        (2) Aerated


CIRCULAR
OVAL
ELL
HORSESHOE

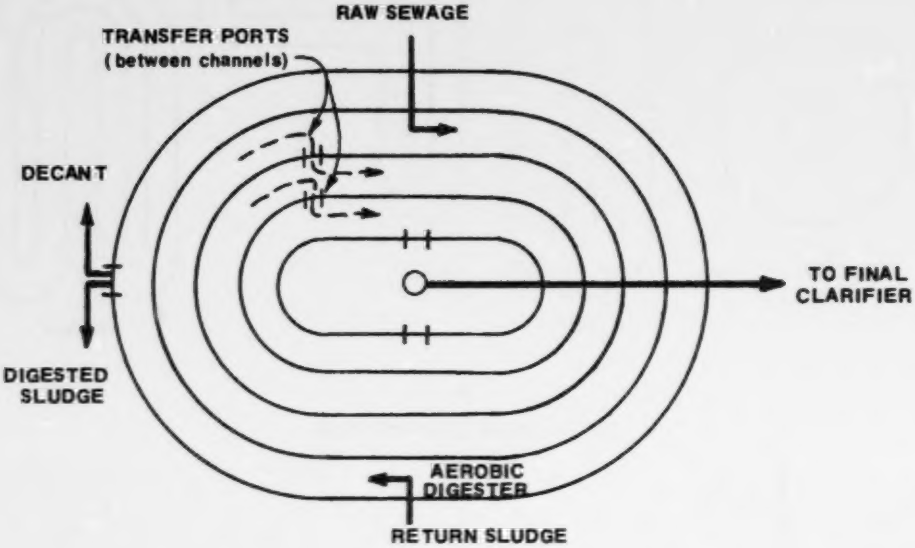
TYPICAL CHANNEL CONFIGURATIONS



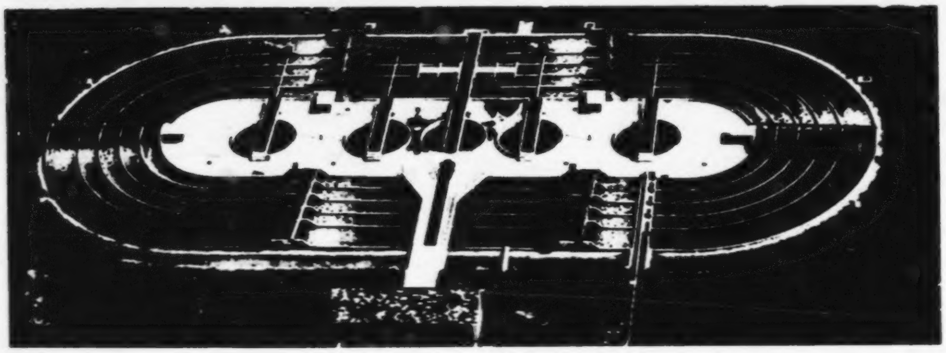
TYPICAL PLANT

(Photo courtesy Lakeside Equipment Corporation)

Figure 2. Single channel oxidation ditch plant.


RAW SEWAGE
TRANSFER PORTS
(between channels)
DECANT
DIGESTED
SLUDGE
TO FINAL
CLARIFIER
AEROBIC
DIGESTER
RETURN SLUDGE

TYPICAL SCHEMATIC (WITH AEROBIC DIGESTION)



TYPICAL PLANT

(Photo courtesy Envirex, Inc.)

Figure 3. Multiple concentric channel oxidation ditch plant.

    - b. Mechanical grit chamber
        - (1) Non aerated
        - (2) Aerated

4. Raw sewage flow measurement, generally following other pretreatment.
    - a. Orifice
    - b. Weir
    - c. Parshall Flume

Typical arrangements generally conform to the following.

1. Coarse screening followed by flow measurement.

2. Coarse screening, grit removal, and flow measurement.

3. Comminutor followed by flow measurement. If the comminutor is not preceeded by a grit chamber, a gravel trap and floatable trap is generally provided prior to comminution.

4. Grit removal, comminutor, and flow measurement.

The most common arrangement observed during the study was gravel trap, comminution, and flow measurement.

Primary sedimentation is practiced at very few oxidation ditch plants and would be considered an unusual variation.

OXIDATION DITCH PLANT ARRANGEMENTS AND DESIGNS

Most manufacturers have developed plant design criteria and suggested plant layouts with a great deal of flexibility in design of plants so that many design variations are possible to meet special treatment requirements or other special conditions.

The simplest arrangement is a single channel plant which is operated to serve as an aeration ditch and final clarifier. Raw sewage flows into the ditch with the aerator operating, and the effluent weir or valve is positioned so no effluent flows out. When the maximum ditch operating level is reached, the aerator is stopped. The raw sewage continues to enter the ditch or is stored in a holding tank. During the time that the aerator is off the ditch is used as a final clarifier and the solids in the ditch settle. When a preset period of time has elapsed the supernatant (effluent) is withdrawn until the ditch level drops to minimum. The supernatant drawoff is completed, the aerator is restarted, raw sewage flows into the plant, and the cycle is repeated.

A variation on this single ditch, intermittent effluent flow arrangement is two ditch intermittent operation. The operation is identical to the single ditch described previously except when one ditch is in the settling and supernatant withdrawal mode, raw sewage is flowing into the other ditch and the aerator is operating. There are many variations to this two ditch intermittent operation but the result is essentially the same.

The configuration most common in the United States is an oxidation ditch followed by a final clarifier. This configuration is operated on a continuous flow basis rather than on a batch basis. Generally, the rate of raw sewage flow into the ditch determines the hydraulic flow through the plant; however, in some cases the rate of mixed liquor withdrawal from the ditch is controlled so the ditch can provide some equalization storage. When the ditch is used to provide equalization storage, the level varies, and a floating or disc type aerator is normally used. This type of flow equalization does not provide a constant flow rate through the plant, but does dampen influent flow variations to the final clarifier. Most oxidation ditch plants in the United States utilize the flow through configuration where the raw sewage flow rate is passed on to the final clarifier. The final clarifier is operated as in any other activated sludge process with settled sludge returned to the oxidation ditch and a portion of the plant sludge is periodically wasted from the system.

The ditch itself can be arranged in almost any configuration as long as it forms a closed circuit. The most common is a single channel circular or oval configuration as shown in Figure 2. The channel can be formed into ells, horseshoes or other configurations to fit the site available. Typical forms are shown in Figure 2. Other plants are designed to utilize multiple concentric, interconnected channels to form the treatment path as shown in Figure 3. In some cases the "island" area on the inside of the ditch is used for the final clarifier in order to conserve space and use "common wall" construction. In some cases this "island" space is reduced in width to a single dividing wall. When a single, vertical dividing wall is used, flow return baffles must be installed around all channel bends. These baffles are vertical walls installed near the center of the channel with a radius of approximately one half the channel width and extending in length around the bend.

Ditch lining can be of reinforced concrete, gunite, asphalt, thin membrane, or unlined. Thin membrane linings have not been used to any extent because the continuous velocities in the ditch have a tendency to lift or move the thin membranes. One ditch lined with what appeared to be ¼ inch thick asphalt impregnated felt has been giving satisfactory service for 6 years. Some asphalt lined ditches in Canada are giving reasonably satisfactory service. Unlined ditches are probably non-existant in municipal applications because of the difficulty in stabilizing such ditches from the effects of erosion. The most common ditch construction, based on information available from the study, is reinforced concrete. Typical construction for sloped side ditches consists of 4 in. of concrete poured against earth backing. The concrete is typically reinforced with welded wire mesh. The sloped side is usually at a 45 degree angle and the concrete is poured relatively dry and carefully finished so that face forms are not required. Gunite can be used to form the ditch lining, but experience indicates the cost to be higher than reinforced concrete for this application. Vertical ditch sides are generally reinforced concrete and must be designed for applicable structural conditions.

Normally, the single channel oxidation ditch is operated as an extended aeration plant. However, more than one ditch could be constructed on one plant site to form several process modifications such as standard activated sludge plus aerobic digestion, two stage nitrification-denitrification, or a three ditch contact stabilization plant.

The multiple concentric channel plant can have any number of channels, but generally from two to five are used with four being common. Channel depth is typically from 4 to 7 feet. These multiple channels are interconnected by submerged ports permitting flow from the outer ring to the inner ring and then to the secondary clarifier. On larger designs, the outer channel(s) can be used as an aerobic digester. In these cases, the raw sewage influent is piped and valved so it can enter any of the first three channels. The return sludge is pumped to either the first or second channel. In all cases the flow of mixed liquor to the final clarifier is from the inner channel. A typical schematic is shown in Figure 3.

The multiple channel arrangement may provide some flexibility over the single channel for certain applications. For instance, some 4 channel multiple channel plants are designed so that they can be operated in the extended aeration mode, standard activated sludge plus aerobic digestion mode, or contact stabilization mode simply by resetting gates and valves. It is also possible to tailor the aeration to the requirements of each channel. This could provide single stage biological nitrification-denitrification as will be discussed in Section 4. The multiple channel arrangement divides a large basin into efficient sized channels to minimize turbulence and short circuiting and provides a maximum sized basin within a minimum land area.

The drawoff of mixed liquor effluent from the aeration system of some oxidation ditch plants is controlled by a submerged port or a narrow weir which provides some equalization of short duration flow variations by allowing a varying level in the aeration channels with varying raw sewage flows. The effectiveness of this flow equalization depends on the duration of the flow surges because as the channel level increases the flow through the orifice or over the weir will increase. If the flow persists, the level will eventually rise high enough to allow the full flow from the channel, but short duration flows will be partially stored in the channel because of the rising water level. Six to twelve inches of level variation are possible depending on the plant and aerator design.

Aerators are marketed by several manufacturers. Aerators provide oxygen transfer to the wastewater and also impart the horizontal velocity to the channel. This horizontal velocity is imparted by the rotation of the aerator in the upper part of the channel contents much like a horizontal paddlewheel. The "Carrousel" system is slightly different because it uses vertical aerators which impart channel velocity by centrifugal action. The detailed construction of aerators varies by manufacturer, but typically consists of an electric motor drive, speed reducer, and rotor with supports

TABLE 2. TYPICAL SINGLE CHANNEL DESIGNS - OXIDATION DITCH PLANTS

Average Daily Flow		BOD lb Per Day @ 200 mg/l Influent	Channel Volume, cu ft	Channel Liquid Depth, ft	Channel Bottom Width, ft	Channel Water Surf. Width, ft	Channel Overall Length, ft	Channel Overall Width, ft	Rotor Number & Length, ft	Total Drive Motor, Horsepower		RAS Pump Capacity, gpm
mgd	gpm									Operating Requirement	Installed Size	
SHALLOW, 45° SLOPED SIDE & ISLAND WALLS												
0.05	35	83	6,640	4	5	13	92	42	1-4'	2.5	3	70
0.1	70	167	13,280	5	8	18	101	52	1-7'	5.2	7½	140
0.2	139	334	26,560	5	9	19	188	54	2-8'	10.3	15	278
SHALLOW, 45° SLOPED SIDE & VERTICAL WALL												
0.5	348	835	66,400	6	18	24	276	49	2-14'	25.0	40	696
1.0	694	1,670	132,800	6	28	34	374	69	2-24'	50.8	60	1,041
1.5	1,041	2,505	199,200	6	28	34	552	69	3-24'	76.2	90	1,561
2.0	1,389	3,340	265,600	6	40	46	537	93	3-32'	101.7	120	2,082
DEEP, VERTICAL SIDE WALLS												
0.20	139	334	26,560	12	15	15	80	31	1-11'	10.4	15	278
0.50	348	835	66,400	12	18	18	161	37	2-14'	25.0	40	696
1.00	694	1,670	132,800	12	28	28	209	57	2-24'	50.8	60	1,041
2.00	1,388	3,340	256,600	12	40	40	248	81	3-32'	101.7	120	2,082
5.00	3,470	8,350	664,000	12	62	62	473	125	4-54'	259.0	300	5,205
10.00	6,944	16,700	1,328,000	12	62	62	918	125	8-54'	518.0	600	10,410

Channel Design:
Detention: 24 hr
Loading 12.5 lb BOD/1,000 cu ft /day

Aeration Equipment:
OC: 2.35 lb oc/lb BOD applied below 2,000' elevation

Source: Lakeside Equipment Corporation

TABLE 3. TYPICAL MULTIPLE CONCENTRIC CHANNEL DESIGN-OXIDATION DITCH PLANTS

Average Daily Flow, mgd	Aeration Channels		Aerobic Digester Channels		Aeration Time, hrs		Discs, Total All Chambers		Overall Dimensions, Ft	
	No.	Vol, cu ft	No.	Vol, cu ft	All Channels	Aeration Channels Only	No.	Total HP	Length	Width
0.05	2	6,667	0	-	24	-	5	7 1/2	28	round
0.10	2	13,333	0	-	24	-	8	10	39	round
0.15	3	20,000	0	-	24	-	11	15	46	round
0.15	2	6,667	1	4,000	14	8	11	15	37	round
0.20	2	9,333	1	5,333	14	8	14	20	42	round
0.30	2	13,333	1	8,000	14	8	20	30	50	round
0.50	3	24,000	1	14,667	14	8	34	40	120	100
1.0	3	44,000	1	26,667	13	8	67	90	171	131
2.0	3	85,333	1	45,333	12	8	135	160	213	173

Based on:

200 mg/l BOD influent

Envirex literature

and bearings. One manufacturer uses an electro-hydraulic drive to provide easy speed variation.

Most aerators operate within the rotational speed range of 50 to 110 rpm. Detailed construction of each manufacturer's aerator is described in a following portion of this section. Aerators are normally installed without protective covers, but in severe and moderately cold areas where spray from aerators will freeze and accumulate, the aerators are generally fitted with covers or complete enclosures which may be heated. A major consideration is the accumulation of chunks of ice in the ditch which might damage the aerator blades or brushes as the chunks pass through. A baffle at water level upstream of each aerator may be helpful to keep ice chunks out of the aerator.

Final clarifier design and application for oxidation ditch plants is the same as for other activated sludge applications. The design overflow rate should be in the range of 400 to 500 gpd/sq ft at average plant flow and 1,000 to 1,200 gpd/sq ft at peak flows. Clarifier design solids loadings should be 30 lb/day/sq ft with provisions for continuous and relatively uniform removal of sludge. Side wall depths should be at least 12 feet and preferably 12 to 14 feet. Clarifiers should be provided with scum baffles and automatic skimmers. It is desirable to make piping provisions so skimmings can be returned to the aeration ditch.

As a general rule, most small oxidation ditch plants (up to 0.5 to 1.5 mgd) are designed based on a 24 hour aeration period. The larger plants may be designed for shorter aeration periods in the range of conventional activated sludge plants. In these cases, some form of supplemental sludge digestion is usually provided with aerobic digestion being most common.

Assuming a BOD loading of 0.17 lb per capita per day, a detention time of 24 hours, and a flow of 100 gallons per capita per day, the ditch loading would be 12.7 lb BOD_5/day/1,000 cubic feet. Assuming a volatile mixed liquor suspended solids (MLVSS) range of 2100 mg/l to 6300 mg/l the food to micro organism ratio (F/M) would be in the range of 0.10 to 0.034 lb of BOD per day per lb MLVSS respectively. Sludge age (defined as the ratio by weight of aeration system solids to influent solids) under these conditions would be in the range of 10 to 33 days respectively. These criteria result in a conservative design for treatment of domestic sewage.

SLUDGE HANDLING

Sludge handling is required at oxidation ditch plants when a portion of the return sludge is wasted in order to maintain the systems solids balance. Generally, this wasted sludge is highly stabilized when the plant is operated in the extended aeration mode (24 hour aeration detention time). If the plant is operated in the conventional activated sludge mode (6 to 8 hour aeration detention time), additional sludge stabilization such as aerobic digestion may be necessary.

Some extended aeration mode oxidation ditch plants are operated without any formal sludge wasting. Several plants were visited where the operator had not wasted sludge for several years and it was claimed that the plant was able to meet effluent standards of 30 mg/l suspended solids and BOD. Generally, this mode of operation is only marginally satisfactory because the plant is prone to periodic upset of the final clarifier resulting in high final effluent solids.

In most oxidation ditch plants where the extended aeration process is used (16 to 24 hours aeration detention) sludge is wasted directly to open drying beds. In a few cases sludge is wasted directly to tank trucks which spread the liquid sludge on the plant grounds or on adjacent land.

When an oxidation ditch plant is designed and operated as a standard activated sludge process, some form of sludge digestion is normally provided. As discussed later, the larger Envirex plants are designed with an extra channel which can be used for aerobic sludge digestion or as an additional aeration channel. The Paris, Texas plant (Envirex) was the only plant visited where separate aerobic digestion was practiced. All of the other plants were operated in the extended aeration mode without separate sludge digestion.

OXIDATION DITCH PLANT UNIQUE EQUIPMENT SUPPLIERS

Several manufacturers supply equipment for oxidation ditch plants. Much of the equipment is standard to most wastewater treatment applications, however, some is unique to oxidation ditch plants as described in this section.

Each manufacturer has developed design and application information for oxidation ditch plants using their unique aeration equipment.

Characteristics of the various aerators are shown in Table 4. Photographs of typical brush and disc type aerators are shown in Figures 4 and 5. These photographs are reproduced from manufacturer's published literature.

Lakeside Equipment Corporation

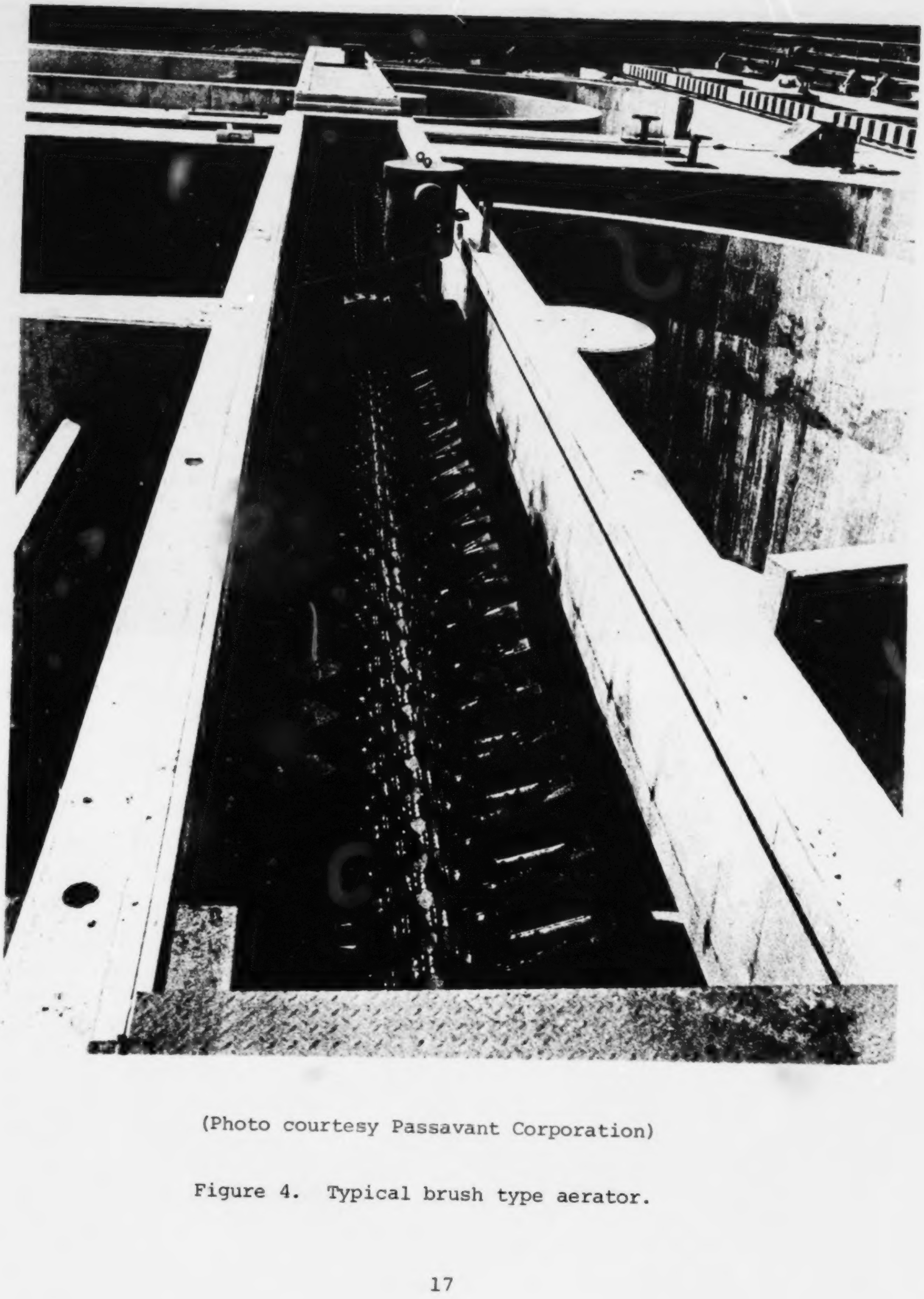
This company was the first to manufacture and market oxidation ditch plant equipment in the USA. Lakeside presently markets a "cage" type aerator and small and large diameter "brush" type aerators (Mini-Magna and Magna Rotors) in addition to other equipment. The cage and Mini-Magna aerators are designed for use in shallow ditches and the Magna Rotor for either shallow or deep ditches. The Magna Rotor is also manufactured in a floating configuration. The immersion depth and speed of the aerators can be changed in order to vary the oxygen transfer rate. The mounting of aerators is normally fixed and the immersion depth is determined by the channel liquid level. This liquid level is adjusted by the weir or other device used to draw off mixed liquor from the channel. Floating aerators have a mechanical means for adjusting immersion by changing the relationship of the pantoons to the centerline of the aerator. Aerator speed is normally factory set by the drive ratio between the motor and the aerator shaft. Variable speed motors can be used, but have seldom been used in the past.

The aerators are driven by electric motors through gear reducer drives.

TABLE 4. COMPARATIVE AERATOR CHARACTERISTICS

Manufacturer	Type or model	Type blades	Diameter, inches	Shaft	Typical submergence range, inches	Typical speed range, rpm	Material of construction	Max. length per shaft, ft
Lakeside	Cage & Mini	Hor. toothed blades	27 1/2	6" OD torque tube	2 to 10	60 to 90	Steel	16
Lakeside	Magna	3" wide brush, 14" long max.	42	14" OD torque tube	4 to 14	50 to 72		30
Passavant	Manmoth Series 5200	3" wide brush, 9.5" long	27	8" OD torque tube	3 to 9.5	90		12
Passavant	Manmoth Series 5300	3" wide brush, 12" long	39 3/8	14" OD torque tube	4 to 12	70		30
Walker	Reelaer Class 6227	3" wide brush, 11" long	38	16" OD torque tube	7 to 10		Steel torque tube galv. blades and hardware	12 min 25 max
Envirex	Disc	1/2" wide plastic disc	52	5" OD torque tube	11 to 21	56 to 58	Steel torque tube	-
Cherne	OTA Aerator	Perforated fiberglass blades	30			up to 110		7 (one size)

SOURCE: Manufacturer's literature



(Photo courtesy Passavant Corporation)

Figure 4. Typical brush type aerator.

(Photo courtesy Envirex, Inc.)

Figure 5. Typical disc type aerator.

The gear reducers are generally of the oil bath, double reduction type and the concentric planetary gear type for cage aerators.

The oxygen transfer capabilities and power requirements for the cage and Magna Rotor aerators are shown in Figures 6 and 7 respectively. All information was taken from manufacturers published literature.

Passavant Corporation

Passavant markets a series 5200 and series 5300 Mammouth aerator both being brush type. The two series are very similar except the outside diameter of the series 5200 is approximately 27 in. and the series 5300 is approximately 39 in. The series 5300 aerator is also manufactured in a floating configuration.

The immersion depth and speed of the aerators can be changed in order to vary the oxygen transfer rate in the same manner as with the Lakeside aerators.

The aerators are driven by electric motors through oil bath, double reduction gear reducer drives.

The oxygen transfer capabilities and power requirements for the Passavant aerators are shown in Figures 8 and 9. All information was taken from manufacturers published literature.

Envirex, Incorporated

The Pacific Flush Tank product line of Envirex includes oxidation ditch plant equipment conforming to the Huisman Orbital System as developed and applied in South Africa. Envirex manufactures aeration discs for use in oxidation ditch plants.

The aeration discs are 4.5 feet in diameter and 1/2 inch thick perforated plastic mounted on line shafts. The two piece discs clamp on to the shaft and can be added or removed in the field. The discs normally rotate at 58.5 rpm through a motor driven gear reducer and final chain drive or at 56 rpm through a motor driven gear reducer directly connected to the disc shaft. Gear reducers are typically, oil bath, double reduction type.

The oxygen transfer capabilities and power requirements for the Envirex discs are shown in Figure 10. All information was taken from manufacturers published literature.

Walker Process

Walker Process Division of CBI markets a horizontal cage surface aerator called the class 6227 ReelAer. This is a brush type aerator with a diameter of approximately 38 in.

The minimum length of the ReelAer is 12 feet and maximum length is

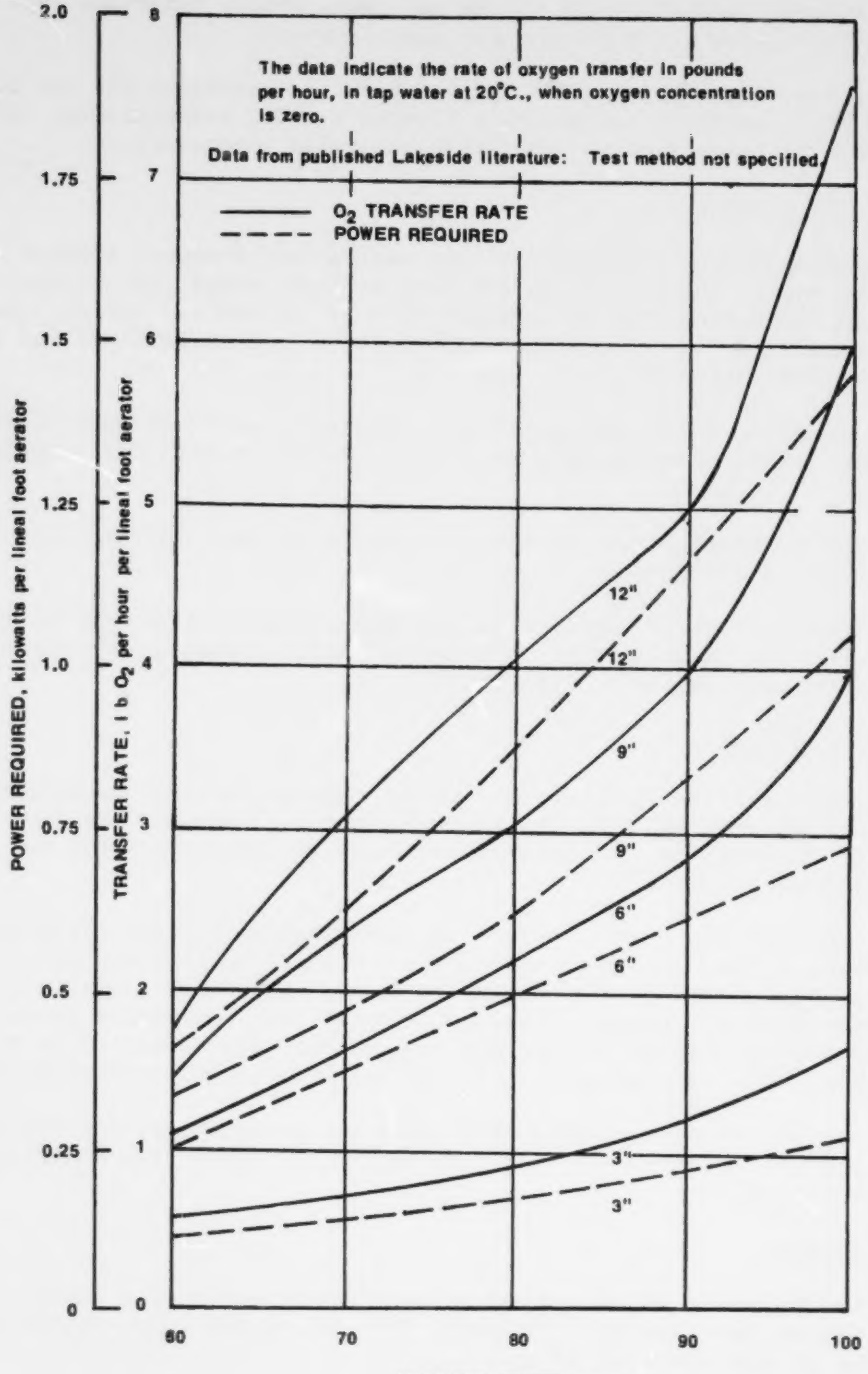
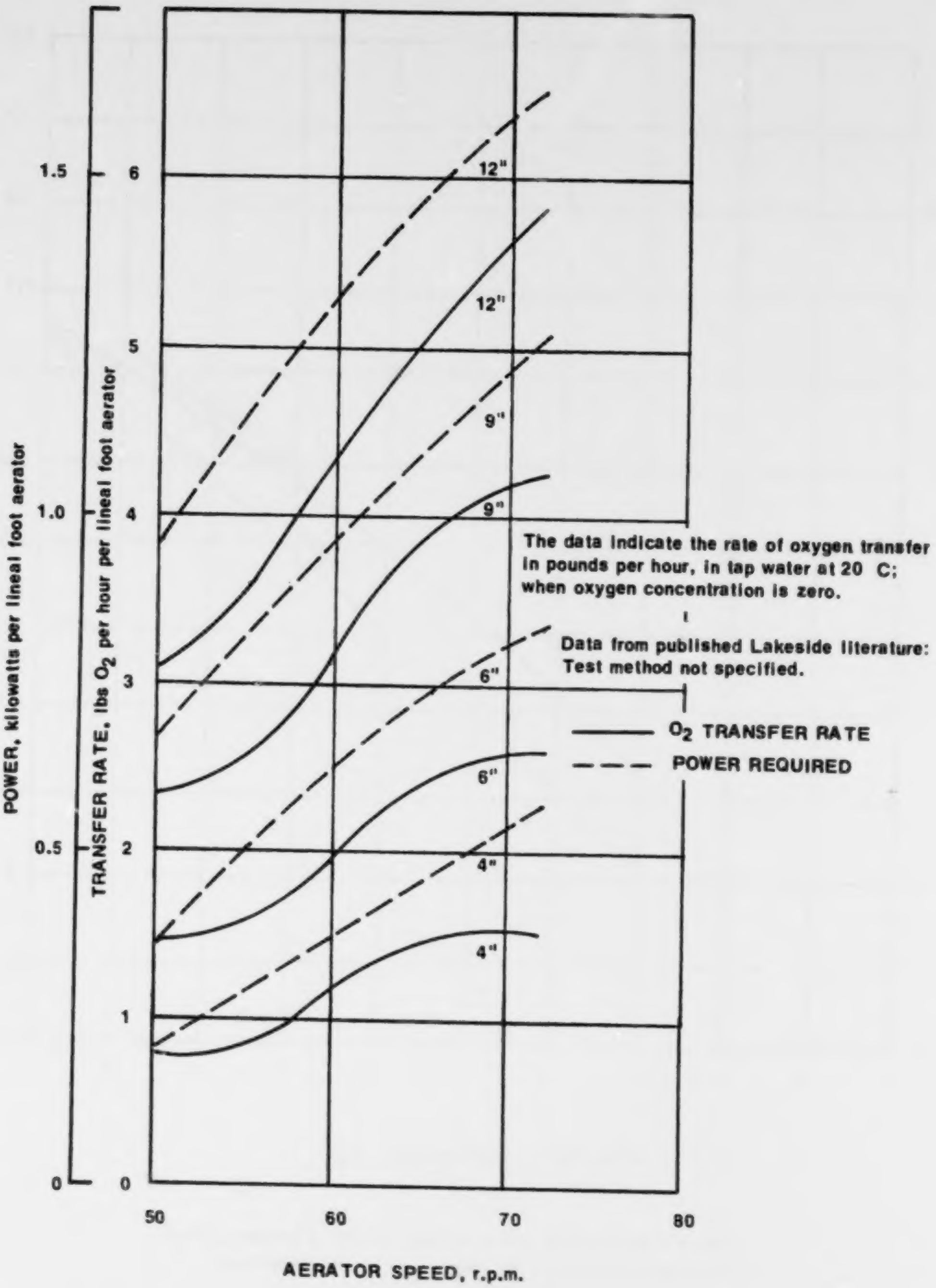


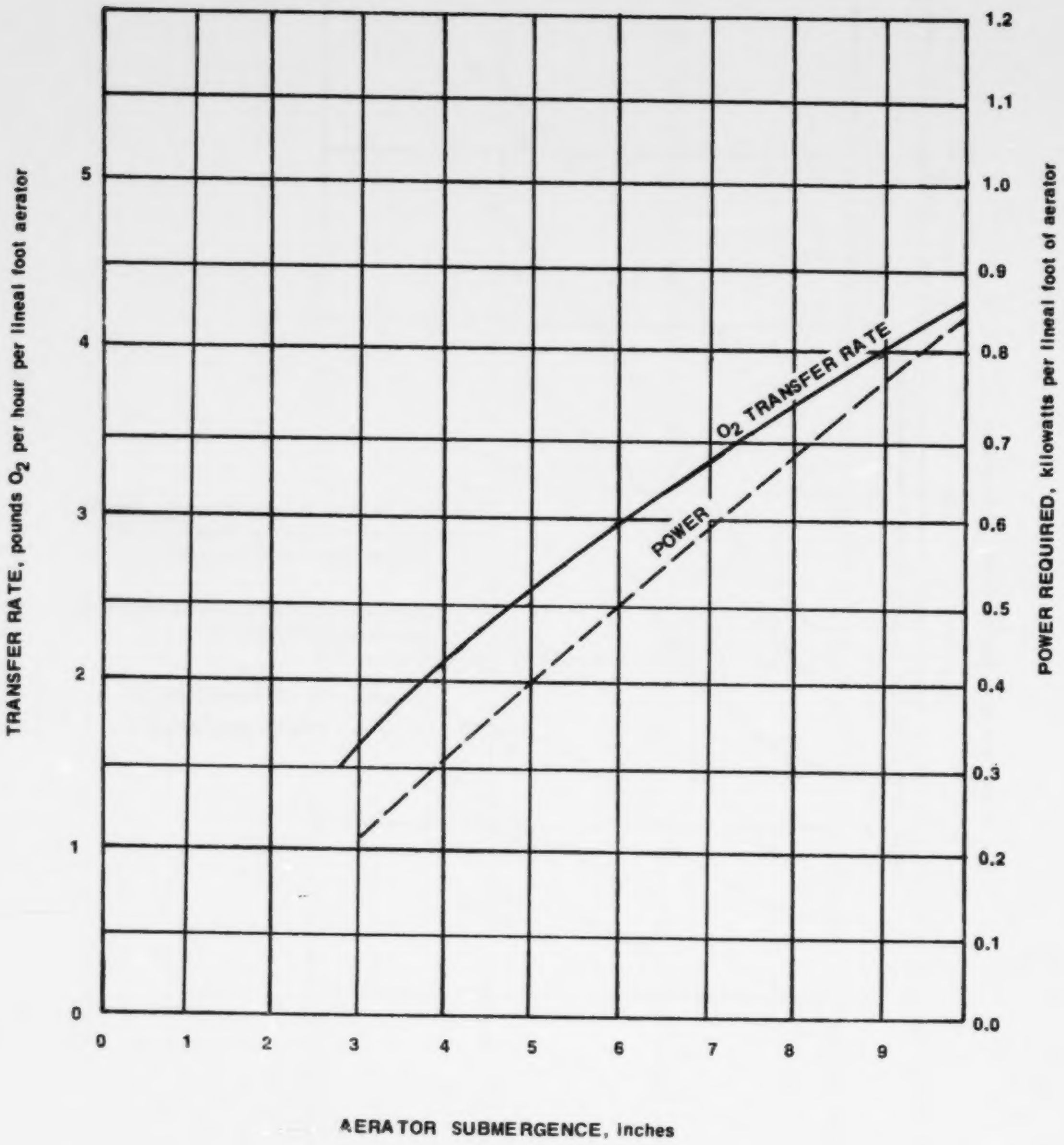
Figure 6. Lakeside Cage and Mini-Magna aerator characteristics.



The data indicate the rate of oxygen transfer in pounds per hour, in tap water at 20 C; when oxygen concentration is zero.

Data from published Lakeside literature: Test method not specified.

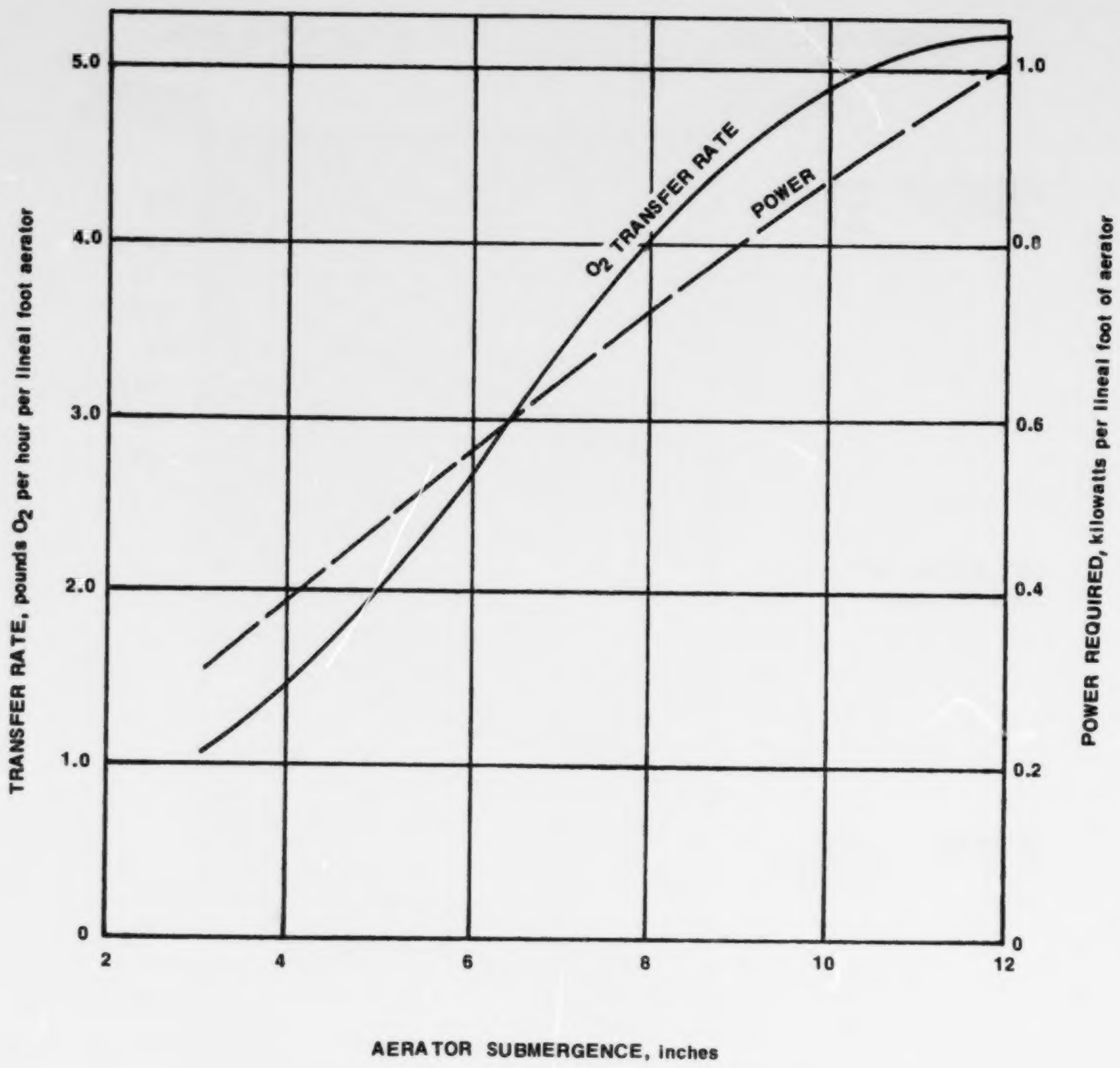
Figure 7. Lakeside Magna-Rotor characteristics.



The data indicate the rate of oxygen transfer in pounds per hour, in tap water at 20°C., when oxygen concentration is zero.

Data from published Passavant literature: Test method not specified.

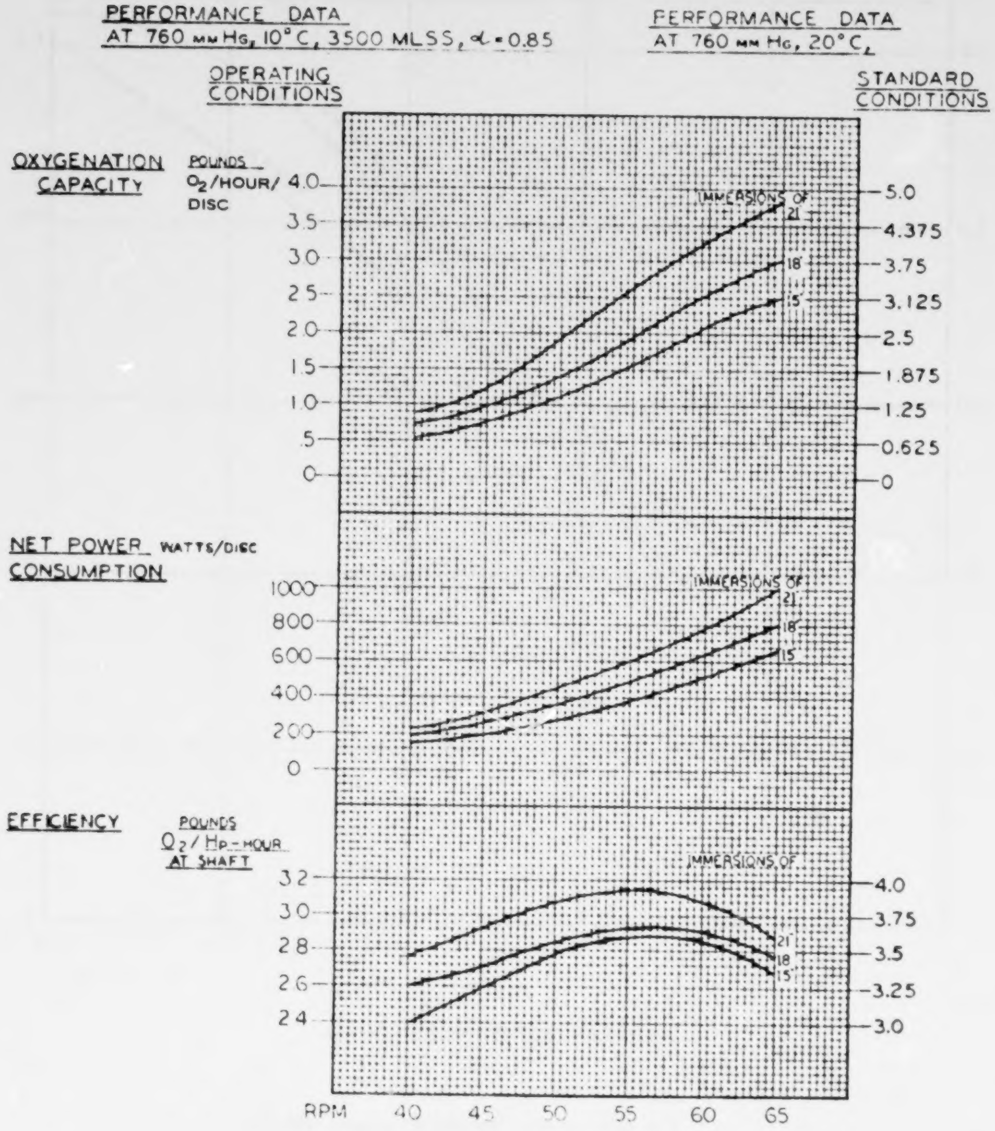
Figure 8. Passavant series 5200 aerator performance, 90 rpm.



The data indicate the rate of oxygen transfer in pounds per hour, in tap water at 20 C., when oxygen concentration is zero.

Data from published Passavant literature: Test method not specified.

Figure 9. Passavant series 5300 aerator performance, 70 rpm.



(Normal range 56 to 58 RPM)

Data from published Envirex literature: Test method not specified.

Figure 10. Envirex 4'-6" aeration disc characteristics.

25 feet. Two aerators may be driven by a single double shaft drive unit. The ReelAer is driven by a direct coupled oil bath, double reduction speed reducer which is belt driven by an electric motor.

Cherne Industrial, Inc.

Cherne manufactures and markets a floating perforated blade type aerator called the "OTA Aerotor", shown in Figure 11, which is suitable for use in oxidation ditches. The aerator is driven by a variable speed hydraulic motor and is capable of speeds up to 110 rpm. A 15 Hp electric motor drives the hydraulic supply system. The unit is available in one size only. The aerator is 30 inches in diameter and 7 feet long and is constructed of reinforced fiberglass. Oxygen transfer varies from 450 pounds of oxygen per day at the low speed range to over 1250 pounds of oxygen per day at the high speed according to Cherne. The oxygen transfer capabilities are shown in Figure 12 based on published literature.

The submergence of the aerator is also adjustable by adding water to the floats to aid in further balancing of oxygen transfer rate to oxygen demand and ditch velocity.

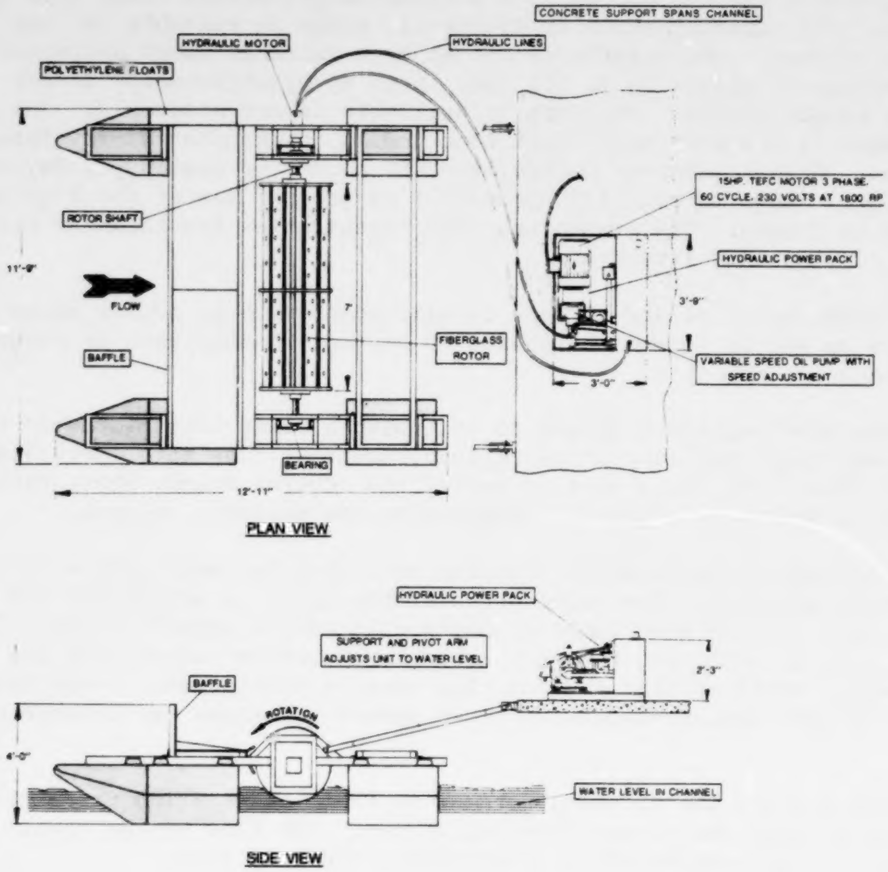
Cherne also markets a device to control the mixed liquor drawoff rate to the final clarifier to provide relative constant flow rate operation of the final clarifier. This unit is called the "Cherne Steady State Control" and normally mounts in a chamber adjacent to the aeration channel.

The "Steady-State Control" consists of 2 circular weir plates with common float mounting. The submergence of the weirs is adjustable and thus the flow rate is adjustable by means of a remote control panel. Once the flow rate is adjusted, the weir assembly elevation varies with the channel water level so that the set flow rate is maintained. A special overflow is provided for excessive flows beyond two times the average daily flow.

Cherne applies the floating aerator so that the aeration ditch can absorb the diurnal raw sewage flow variations. The flow to the final clarifier is then controlled to a relatively constant rate.

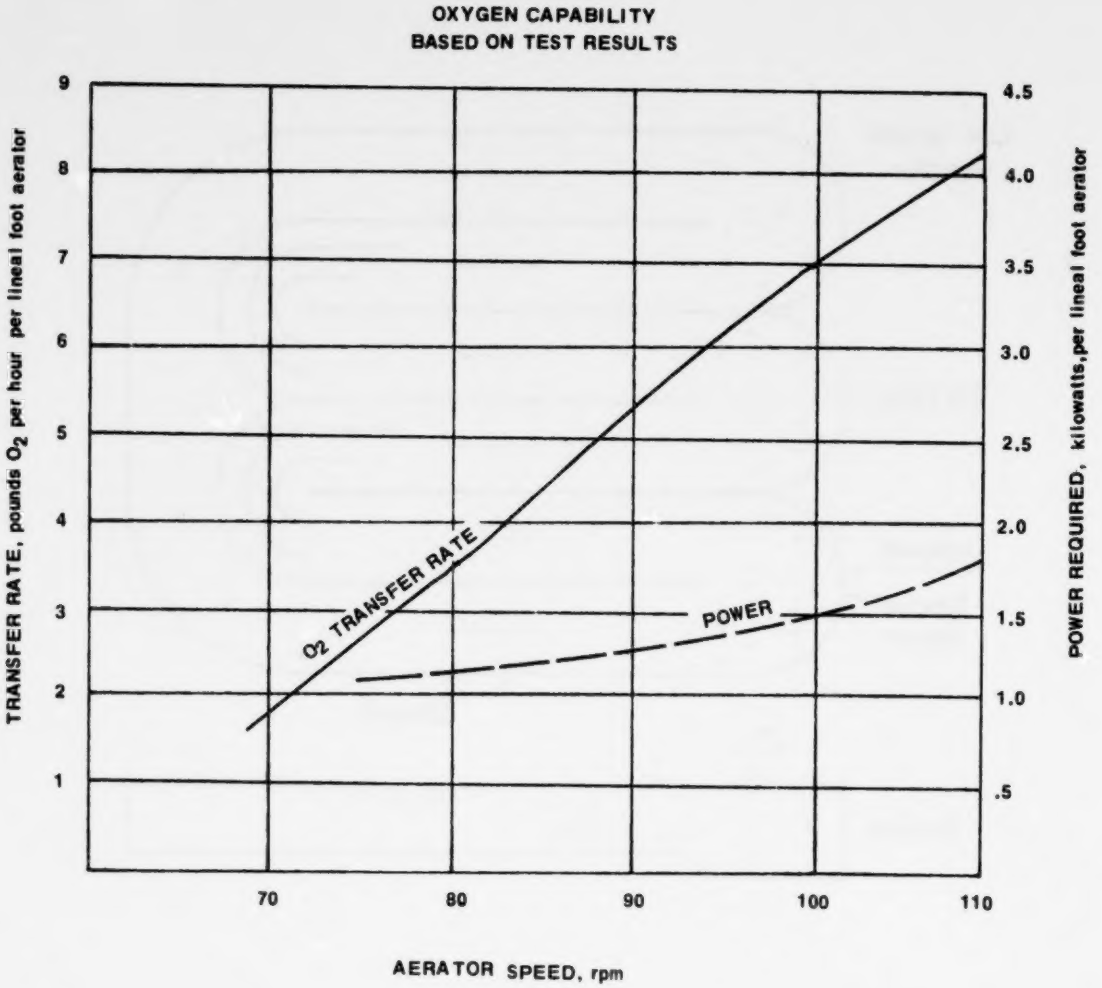
Envirotech Corporation

On June 1, 1976, Envirotech acquired licensing rights to the patented "Carrousel" system for the United States and Canada. These rights include access to the technical staff of Dwars, Heederik en Verhey, B.V., Amersfoort, Dutch engineers who developed the system and to operating data derived from more than 100 installations. The system, as shown in Figure 13, uses low speed stationary vertical mechanical non sparged aerators rather than horizontal brush or disc type aerators and ditches up to 15 feet deep. The vertical aerators are mounted at 180 degree bends in the channel near the dividing wall. The installation is designed so this aerator can impart velocity in the channel. The previous licensee, Envirobic Systems, Inc. did not market plant equipment; but only the right to construct a Carrousel plant, access


CONCRETE SUPPORT SPANS CHANNEL
HYDRAULIC MOTOR
HYDRAULIC LINES
POLYETHYLENE FLOATS
ROTOR SHAFT
15HP. TEFC MOTOR 3 PHASE. 60 CYCLE. 230 VOLTS AT 1800 RP
11'-9"
FLOW
7'
HYDRAULIC POWER PACK
3'-9"
FIBERGLASS ROTOR
BAFFLE
VARIABLE SPEED OIL PUMP WITH SPEED ADJUSTMENT
3'-0"
BEARING
12'-11"
PLAN VIEW
HYDRAULIC POWER PACK
SUPPORT AND PIVOT ARM ADJUSTS UNIT TO WATER LEVEL
2'-3"
BAFFLE
ROTATION
4'-0"
WATER LEVEL IN CHANNEL
SIDE VIEW

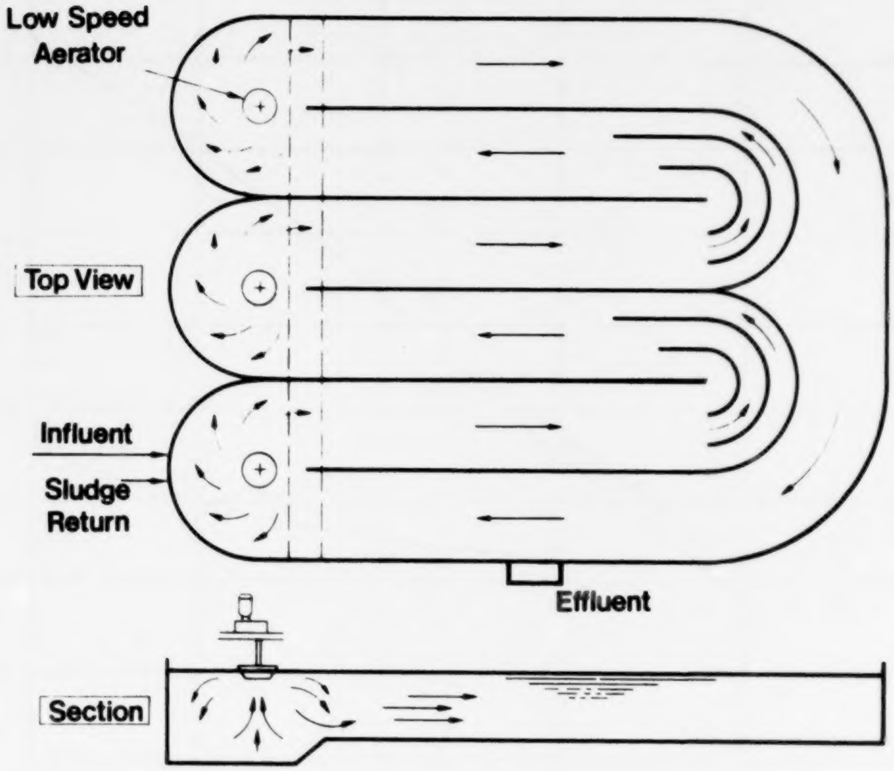
(Courtesy of Cherne Industrial, Inc.)

Figure 11. Cherne "OTA Aerotor" aerator configuration



Data from published Cherne literature: Test method not specified.

Figure 12. Cherne OTA aerator characteristics.


Low Speed Aerator
Top View
Influent
Sludge Return
Effluent
Section

(Courtesy of Envirotech Corporation)

Figure 13. Envirotech "Carrousel" schematic

to technical data, and Carrousel pilot plant equipment. Envirotech will license the design and construction of Carrousel plants. This license fee typically is paid to Envirotech as a bid item in the construction contract. The equipment is procurred by open bid and can be supplied by a number of different manufacturers.

Carrousel literature indicates approximately 139 plants designed or operating in Europe and 154 in the world as of October 1975. One Carrousel plant treating tannery waste has been in operation in Winchester, New Hampshire since January 1977. Approximately six others are under design in the United States.

EVALUATION OF DESIGNS

Pretreatment

Screening seems to be the single most important pretreatment step. If rags, boards, and other similar objects are not removed prior to the ditch they will cause trouble with the aerators and will, in most cases, plug sludge control valves (telescoping valves), sludge lines, sludge pumps, and weirs. Few problems were observed or reported directly related to inadequate grit removal although it is sure that this grit accumulates in the ditch and will have to be removed at some time. Grit was not a significant operational consideration for oxidation ditch plants up to 1 mgd or more in size.

Aerator Design and Applications

The characteristics of each manufacturers aerator is shown in Table 4. Based on a review of the physical characteristics of the aerators it is difficult to see any significant difference between the Lakeside Magna , the Passavant Series 5300, and the Walker ReelAer aerators. The Lakeside Mini-Magna, Cage, and Passavant Series 5200 aerators are also very similar.

For evaluation purposes aerators are classified in the following categories:

1. Small diameter horizontal aerators
   a. Lakeside Cage
   b. Lakeside Mini-Magna
   c. Passavant Series 5200
   d. Cherne OTA Aerotor
2. Large diameter horizontal aerators
   a. Lakeside Magna
   b. Passavant Series 5300
   c. Walker ReelAer
3. Envirex 4'-6" disc aerators

All of the aerators will operate over a variable submergence, thus allowing some variation of level within the ditch. Cherne, Passavant, and Lakeside supply floating aerators which would be applicable in cases where substantial variations in level were expected, where it might be desireable to maintain constant aeration submergence with a varying ditch water level, or where the flexibility of a movable aerator is needed.

The choice between aerators in the small and large diameter horizontal aerator groups is largely one of individual preference and, in most cases, will be determined by the bidding situation. Generally, the small diameter horizontal aerators are limited to use in the shallower ditches and the large diameter are used in shallow or deep ditches.

The large diameter horizontal aerators are available in lengths up to 30 feet. Field experiences with deformation and failure of torque tubes would suggest care in specifying the longer aerators.

The Envirex disc aerator is somewhat unique to other manufacturers equipment and to date, in the U.S., has been used only in the multiple concentric channel type plant as far as the authors can determine. Disc aerators could be used in single channel type plants. The discs may offer some advantage over the other types of aerators. Discs can easily be added to or deleted from the shaft in the field because they come in two halves and clamp to the shaft. This allows the aeration to be modified in the field.

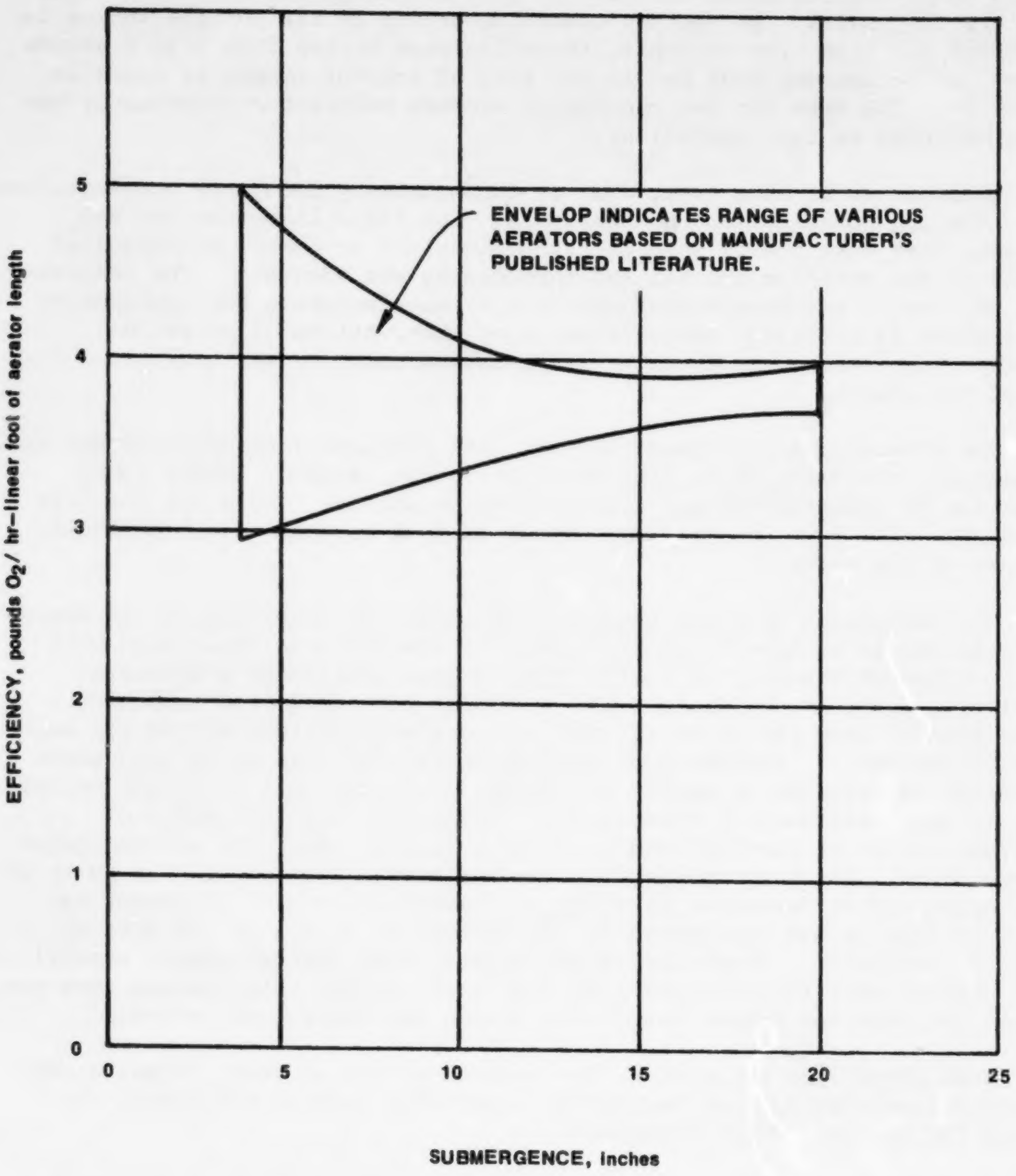
The Cherne OTA Aerotor does have a standard built-in variable speed feature which may be of advantage in some cases. The other manufacturers could supply variable speed motors or drives in all probability, but this feature was not found at any operating facilities.

There are many available configurations of plants and aerators and it is recommended that care be taken to select the most energy efficient configuration because aerator energy is a major operating expense for oxidation ditch plants. There should be little actual difference between manufacturers aerator power requirements for given size plants. Some variation was noted, however, and this may be because the illustrative plant designs were not completely optimized for each manufacturers equipment and design recommendations.

Analysis of energy requirements from actual operating plants showed no consistent advantage for a particular manufacturers equipment.

A composite range of aerator oxygen transfer efficiency under standard conditions is shown in Figure 14. These transfer tests were conducted by various manufacturers and laboratories at different times, in different facilities, but under similar stated test conditions. There is no easy way to evaluate the comparability of the results between aerator manufacturers, so the design engineer must use his judgement or will have to run independent tests.

Figure 14 is based on manufacturers published performance information



Assumptions:

1. Tapwater, 20° C, starting O_2 concentration = 0
2. Based on manufacturer's published literature shown in figures 6 through 11.

Figure 14. Horizontal aerator efficiency.

and is non specific; being intended to show the range for horizontal type aerators in general. At typical operating depths of six to nine inches for the brush and cage type aerators, the efficiency varies from 3 to 5 pounds oxygen per horsepower hour per lineal foot of aerator length as shown in Figure 14. The data are not consistent between manufacturers probably due to differences in test conditions.

There seems to be a great deal of inconsistency among the manufacturers' data relating to performance of aerators. The field investigation did indicate that most plants were able to maintain 2 to 6 mg/l of dissolved oxygen in the aeration channel and that mixing was adequate. The information and design recommendations provided by manufacturers for application of aerators is generally adequate based on observations of operating plants. There was no indication that aeration capacity was undersized for any of the plants.

The site visits confirmed that the real problems (and differences) are mechanical; the drive unit, line bearings, seals, aerator torque tube, materials of construction and similar considerations. These are the features that will have the greatest impact on long term operation and maintenance of the plant.

The mechanical features should receive careful consideration in design and preparation of specifications. Most of the bearings associated with aerators are of the ball or roller type; grease lubricated with seals. Therefore, bearings and drive units should be protected from splashing water both to keep the water off and to provide convenient access for maintenance personnel. Extension of bearing grease fittings up to convenient locations on catwalks or easily accessable locations will certainly contribute to more satisfactory maintenance. Means for field alignment of bearings should be provided and bearings should be carefully aligned prior to operation. The bearing supports for horizontal aerators may be up to 30 feet apart and differential settling with eventual loss of alignment may occur if this is not considered in the structural design of the aerator support foundation. Generally, bearings with some self alignment capability will provide more satisfactory long term service, but this feature does not reduce the need for proper structural design and initial alignment.

Some means must be provided for removal of the aerator; either a permanently installed lifting mechanism, a portable lifting mechanism, or access for mobile lifting equipment.

Many problems were caused by the aerator "slinging" mixed liquor onto adjacent structures and onto aerator bearings, couplings, seals, and drive units. Most aerators have "slingers" at each end, but they only compound the problem when the wind is blowing as liquid flying off the "slinger" is blown by the wind. Mixed liquor which lands on adjacent walkways causes ice formation when it is cold and algal growths cause slippery conditions at other times. Bearings and drive units should be adequately shielded from the inevitable liquid spray.

Consideration should be given to installation of at least two aerators in each ditch so that a standby is available when troubles occur.

An adequate stock of spare parts for aerators and other equipment should be maintained or readily available to plant personnel so down time is minimized. This includes aerator bearings, seals, couplings, and belts. Additional information is provided in Section 5.

Evaluation of Oxidation Ditch Plant Design Procedures

The oxidation ditch is an unusual process because of the aeration basin geometry, but results from operating plants indicate consistently high performance results without any more than average operator attention. Costs are equal to or less than competing treatment processes.

The long narrow aeration channel of the oxidation ditch plant provides a complete mix activated sludge process. Even though a plug flow mode would seem to be applicable, the minimum velocity of 1.0 fps results in a cycle time of less than 15 minutes in the longest channels used. Compared to the typical 8 to 24 hour detention time design, the channel circuit time becomes insignificant. Therefore, the oxidation ditch may be considered a completely mixed activated sludge process (CMAS) with respect to organic load. It is not completely mixed with respect to dissolved oxygen profile because this profile can vary significantly around the channel and vertically within the channel. The mixed liquor can pass through varying oxygen rich or deficient zones as it passes around the channel, however, which is important when considering nitrogen removal as discussed in Section 4.

The procedure for design of an oxidation ditch plant is basically the same as used for an extended aeration process with emphasis given to the hydraulic considerations imposed by the basin geometry. The following is a rational design procedure which is consistent with manufacturers design procedures. The manufacturers can provide assistance in design for specific applications.

Some low alkalinity wastewaters may require pH adjustment when subjected to extended aeration.

Oxidation ditch plants are designed for long sludge retention times and nitrification will occur if sufficient oxygen is provided. This design procedure assumes that sufficient oxygen will be provided for complete nitrification in addition to satisfying the carbonaceous BOD requirements. Complete mix activated sludge plant design procedures have been described by McKinney(1), Monod(3) and Eckenfelder(2) and presented in a unified model by Goodman(4,5). It is frivolous and time consuming to compare the nuances of these various models; proper use of any of these models results in sufficiently accurate results. General relationships derived from the unified model(4) will serve the purposes of this discussion.

Carbonaceous BOD stabilization requires oxygen for conversion of the organics to CO_2, water, and bacterial cells. In addition, oxygen for endogenous respiration of the cells is required, depending on the solids retention time (SRT) which is defined as the ratio by weight of aeration system solids to wasted solids (waste sludge plus effluent solids). A generalized relationship between lb oxygen/lb BOD versus SRT for a 24 hour detention basin is shown in Figure 15.

As the SRT approaches 20 days the oxygen requirement becomes asymptotic to the value, 1.25 lb oxygen/lb BOD_5 applied and the oxygen for nitrification is approximately equal to 4.5 lb of oxygen per lb of ammonia nitrogen oxidized. Therefore, the total oxygen requirement may be calculated as follows.

$$\text{lb oxygen} = 1.25 \times \text{lb } BOD_5 + 4.5 \times \text{lb } NH_3\text{-}N$$

For example, the oxygen requirements for removal of 200 mg/l of BOD_5 and 30 mg/l of ammonia nitrogen from a one mgd flow of wastewater are:

$$BOD = 200 \times 8.33 \times 1 = 1667 \text{ lb/day}$$

$$NH_3\text{-}N = 30 \times 8.33 \times 1 = 250 \text{ lb/day}$$

$$\text{Oxygen Required} = 1667 \times 1.25 + 250 \times 4.5 = 3209 \text{ lb/day}$$

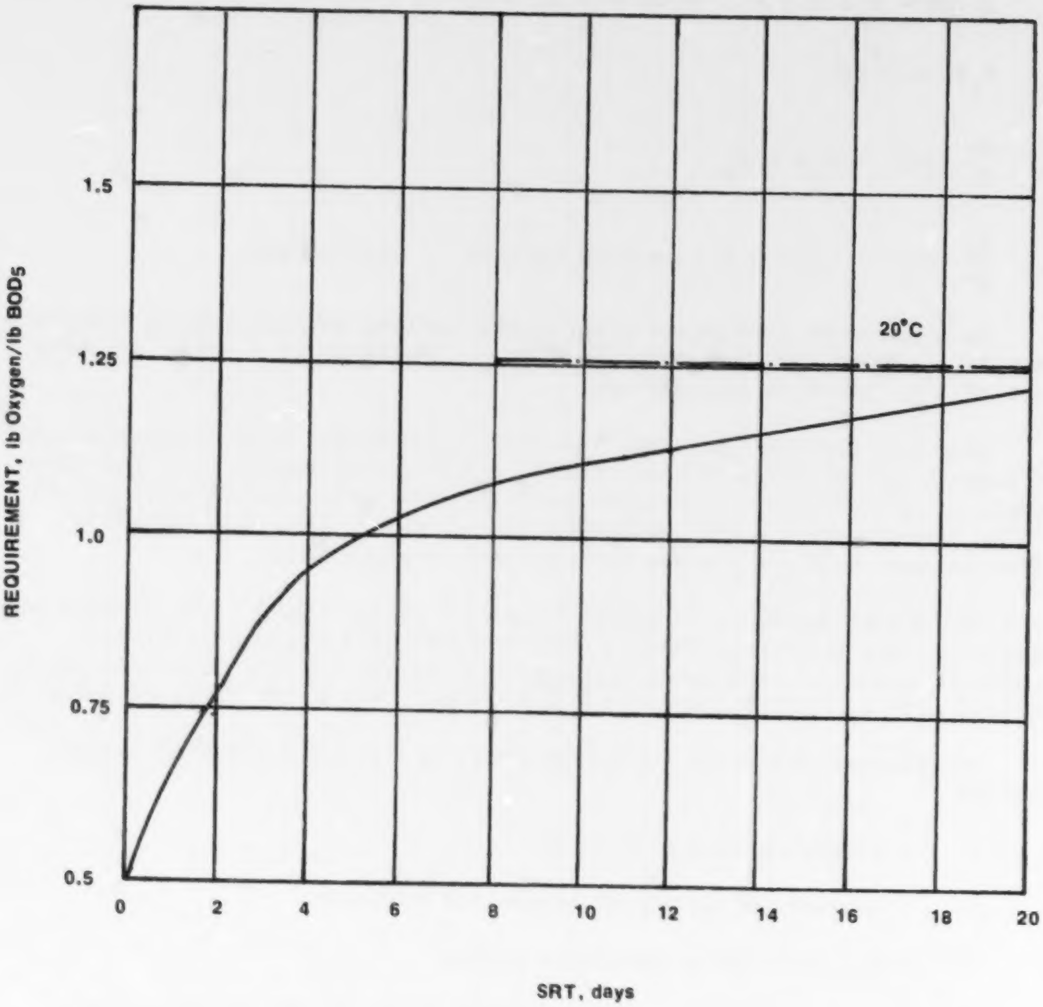
The manufacturers typical design procedure for circulation would provide a 1 million gallon basin and 48 lineal feet of aerator. The aerator(s) would be driven by a combined power of 60 horsepower and would operate at approximately 51 horsepower. This is comparable to conventional activated sludge requirements.

If the following design conditions were established:

Minimum Basin Dissolved Oxygen	2.0 mg/l
Elevation	500 ft
Alpha α	0.8
Beta β	0.95
C_s	oxygen saturation concentration, mg/l
T	20C
K_La	mass transfer coeff.

The calculated oxygen transfer capability of this aerator to pure water at standard conditions would be:

$$\frac{do}{dt} = \alpha K_La \times 1.024^{(T-20)} (\beta C_s - C)$$



Based on calculations using the unified model (4).

Figure 15. Carbonaceous BOD oxygen requirements.

$$\frac{do}{dt} \text{ required} = \frac{3209 \text{ lb/day}}{24 \times 1 \times 8.33} = 16.05 \text{ mg/l/hr}$$

$$16.05 = 0.8\ K_La \times 1\ (0.95 \times 9.1 - 2.0)$$

$$K_La = 3.03$$

$$\frac{do}{dt} \text{ STD} = K_La \times C_s$$

$$\frac{do}{dt} \text{ STD} = 3.03 \times 9.2 = 27.9 \text{ mg/l/hr} = 5578 \text{ lb/day}$$

Manufacturers literature indicates a typical average oxygen transfer rate of 5 pounds per hour per lineal foot of aerator or a total transfer rate of 5760 lb of oxygen per day.

This calculation indicates that for the average design condition the 48 lineal feet of aerator will provide the required oxygen. If the oxygen supply is not adequate to satisfy both the carbonaceous BOD_5 and nitrification demands under certain operating conditions, less than complete nitrification will take place during these conditions.

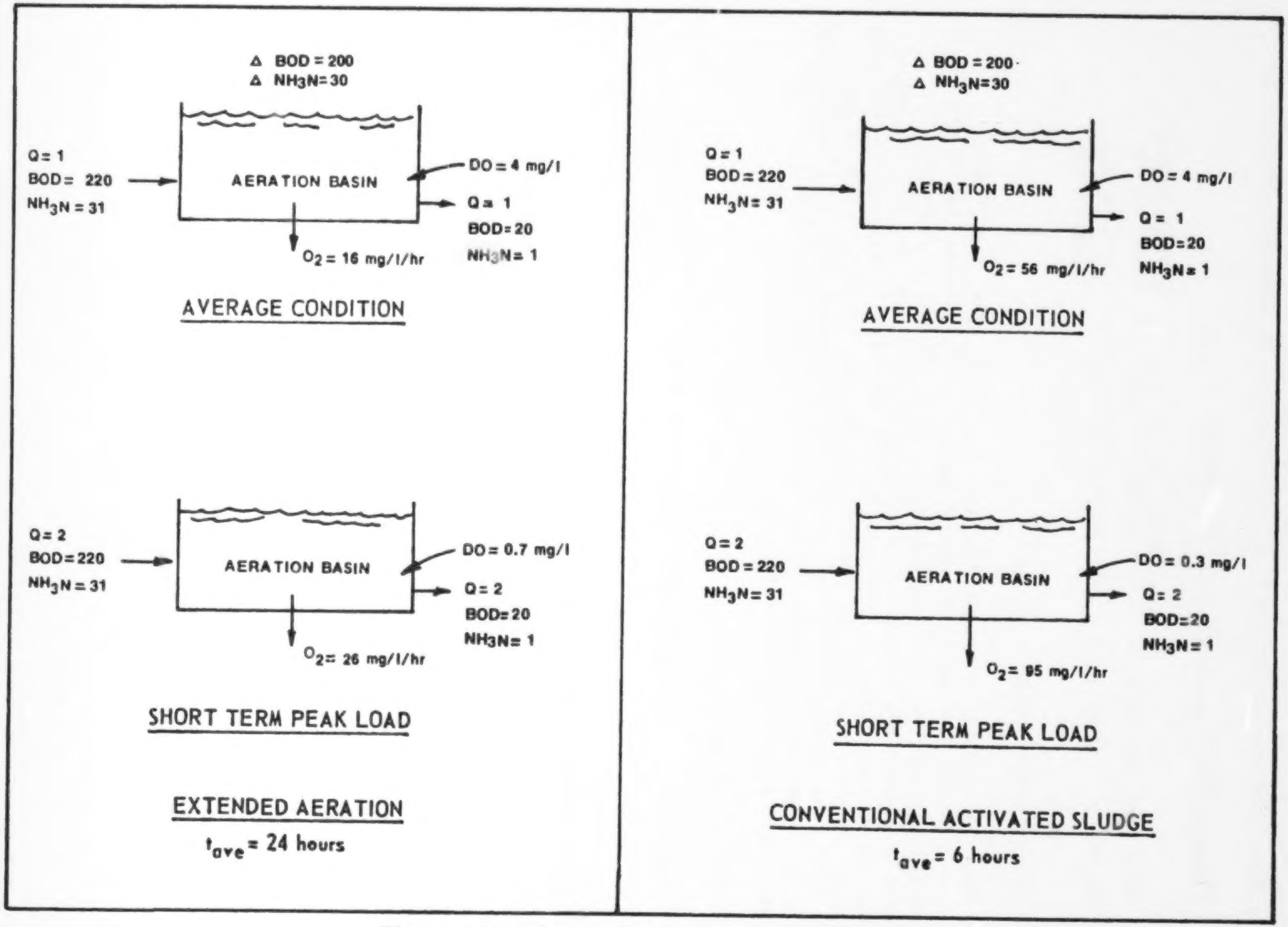
The design basis of 24 hours detention is not sacred. It appears the origin of this detention time is from the extended aeration activated sludge process and represents a convenient convention for municipal applications with a typical loading of 12.7 lb BOD_5 per 1,000 cu ft of aeration.

The process needs for oxidation ditch or extended aeration plants include:

1. A stable operating process
2. Conservative supply of oxygen for process
3. Stabilized waste activated sludge

The stable operating process requirement is served by providing a sufficiently large aeration basin that peak flows will not cause extreme variations in oxygen demand or unusual shifts in solids inventory between the aeration basin and sedimentation basin.

For small plants, variations in organic and hydraulic loading are generally more extreme. However, aeration basin detention time will have only a modest effect on the percent increase in oxygen uptake rate caused by short duration organic load variations. A comparison between a conventionally designed 6 hour detention aeration basin and a 24 hour detention aeration basin is shown in Figure 16 for an average loading condition and a short term peak load condition. The short term peak load imposed



Δ BOD = 200
Δ NH3N=30
Q = 1
BOD = 220
NH3N = 31
AERATION BASIN
DO = 4 mg/l
Q = 1
BOD=20
NH3N= 1
O2 = 16 mg/l/hr
AVERAGE CONDITION
Q = 2
BOD=220
NH3N = 31
AERATION BASIN
DO = 0.7 mg/l
Q = 2
BOD=20
NH3N= 1
O2= 26 mg/l/hr
SHORT TERM PEAK LOAD
EXTENDED AERATION
tave = 24 hours
Δ BOD = 200
Δ NH3N=30
Q = 1
BOD = 220
NH3N= 31
AERATION BASIN
DO = 4 mg/l
Q = 1
BOD=20
NH3N= 1
O2= 56 mg/l/hr
AVERAGE CONDITION
Q = 2
BOD = 220
NH3N= 31
AERATION BASIN
DO = 0.3 mg/l
Q = 2
BOD=20
NH3N = 1
O2= 95 mg/l/hr
SHORT TERM PEAK LOAD
CONVENTIONAL ACTIVATED SLUDGE
tave = 6 hours

Figure 16. Effect of organic variations.

represents a sudden doubling of flow, and mass BOD_5 and ammonia. The following illustrates the assumptions and calculations in developing Figure 14.

For a 1 mgd flow and 24 hour detention

				Mass				Oxygen Demand
BOD_5	=	200 mg/l	=	1,667 lb/day	x	1.25	=	2,084 lb/day
NH_3-N	=	30 mg/l	=	250 lb/day	x	4.5	=	1,125 lb/day
								3,209 lb/day

Hourly uptake rate = 16 mg/l

When the load to plant is doubled (Q = 2 mgd, BOD_5 = 220 mg/l, NH_3-N = 31 mg/l) the immediate oxygen demand will increase due to the incremental increase in load consisting of the conversion of ammonia to nitrate and synthesis of organics. The short term load change will have little immediate effect on the basin active bacterial fraction because this fraction was based on average loading and can not increase immediately. Therefore, the oxygen demand will reflect only synthesis of the incremental BOD. The immediate increased oxygen demand for the incremental increase in mass load is calculated as follows:

Nitrification	= 250 lb/day more x 4.5	= 1,125 lb/day
Synthesis	= 1,670 lb/BOD_5/day more x 0.5	= 835 lb/day
		1,960 lb/day more

Total oxygen demand:	Base	3,209 lb/day
	Increment	1,960 lb/day
		5,169 lb/day = 26 mg/l/hr

Assume 4 mg/l dissolved oxygen at 16 mg/l/hr

$$KLA = \frac{16}{\alpha\ (\beta Cs-4)}$$

$$= \frac{16}{0.8\ (0.95 \times 9.1-4)}$$

$$= 4.3$$

At 26 mg/l/hr

$$C = 9.1\ \beta - \frac{26}{\alpha\ KLA}$$

$$= (9.1)\ (0.95) - \frac{95}{(0.8)\ (15)}$$

$$= 0.7 \text{ mg/l}$$

$$\Delta = \frac{3.3 \text{ mg/l}}{10 \text{ mg/l/hr}}$$

$$= 0.33 \text{ hr or } 19 \text{ min}$$

For a 1 mgd flow and 6 hour detention

SRT = 5 days

lb O_2/lb BOD_5 = 1.0

MASS Oxygen Demand

BOD_5 = 200 mg/l = 1670 lb/day x 1.0 = 1670 lb/day

NH_3-N = 30 mg/l = 250 lb/day x 4.5 = 1125 lb/day

2795 lb/day

Hourly uptake rate = 56 mg/l/hr

Incremental doubling of load

Oxygen Demand

Nitrification = 250 lb/day x 4.5 = 1,125 lb/day

Synthesis = 1670 lb/day x 0.5 = 835 lb/day

1,960 lb/day

Total oxygen demand: Base 2,795 lb/day
1,960 lb/day
4,755 lb/day = 95 mg/l/hr

Assume 4 mg/l dissolved oxygen at 56 mg/l/hr

$$KLA = \frac{56}{\alpha\ (\beta C-4)}$$

$$= \frac{56}{0.8\ (0.95 \times 9.1-4)}$$

$$= 15$$

At 95 mg/l/hr

$$C = 9.1\ \beta - \frac{95}{\alpha\ KLA}$$

$$= (9.1)\ (0.95) - \frac{95}{(0.8)\ (15)}$$

$$= 0.3 \text{ mg/l}$$

$$\Delta = \frac{3.7\ \text{mg/l}}{39\ \text{mg/l/hr}}$$

$$= 0.095\ \text{hr or } 5.7\ \text{min}$$

The 24 hour detention basin (at average flow) would experience a 63 percent increase in oxygen demand, from 16 mg/l/hr to 26 mg/l/hr. If the oxygen concentration in the basin were 4 mg/l, it would eventually drop to 0.7 mg/l at the higher uptake rate and have a 3.3 mg/l buffer, or at least 18 minutes at the increased uptake rate for the excess basin dissolved oxygen to absorb the added load.

The 6 hour detention basin (at average flow) experiences a 70 percent increase in oxygen demand, from 56 mg/l/hr to 95 mg/l/hr. If the oxygen concentration in the basin were 4 mg/l, it would eventually drop to 0.3 mg/l and would have a 3.7 mg/l buffer which represents 6 minutes to absorb the added load at the increased uptake rate.

The greater detention period will result in a slightly more stable system for short term variations in organic load.

Most plant upsets are caused by loss of solids from the final clarifier either by poor solids inventory management or marginal designs. The use of longer detention periods provides significant advantages in maintaining good quality effluent with variations in hydraulic load. An example is shown in Figure 17. The calculations for Figure 17 are simplified by assuming no sludge wasting, no loss of solids in the effluent during the transition period, and a constant sludge return rate. It is also assumed that the flow increase occurs after a period of stable operation at average flow. In reality, most such increases would be after a period of stable operation at substantially less than average flow.

Most small plants are operated with a set, or fixed, return sludge flow rate. Typically, small plants are designed for a maximum return sludge flow rate of 100 percent of the average daily plant flow and operated at an average of 30 to 50 percent return sludge flow rate.

The comparison shown is between an aeration basin having 24 hours detention versus an aeration basin having 6 hours detention both with conventionally designed sedimentation basins. At night, when influent rates are low, the system solids tend to shift to the aeration basin since the solids flux to the final sedimentation basin is low and the recycle rate is constant. When the daily peak flows occur, the solids shift to the final sedimentation basin. The critical consideration is preventing the filling of the final sedimentation basin with solids and then spilling over into the effluent. The example in Figure 17 depicts the percentage of the final sedimentation basin which is used for solids storage.

The 24 hour detention aeration basin under typical operating conditions will result in only 18 percent of the volume of the final sedimentation basis occupied by sludge. A sudden increase in flow (2 times average)

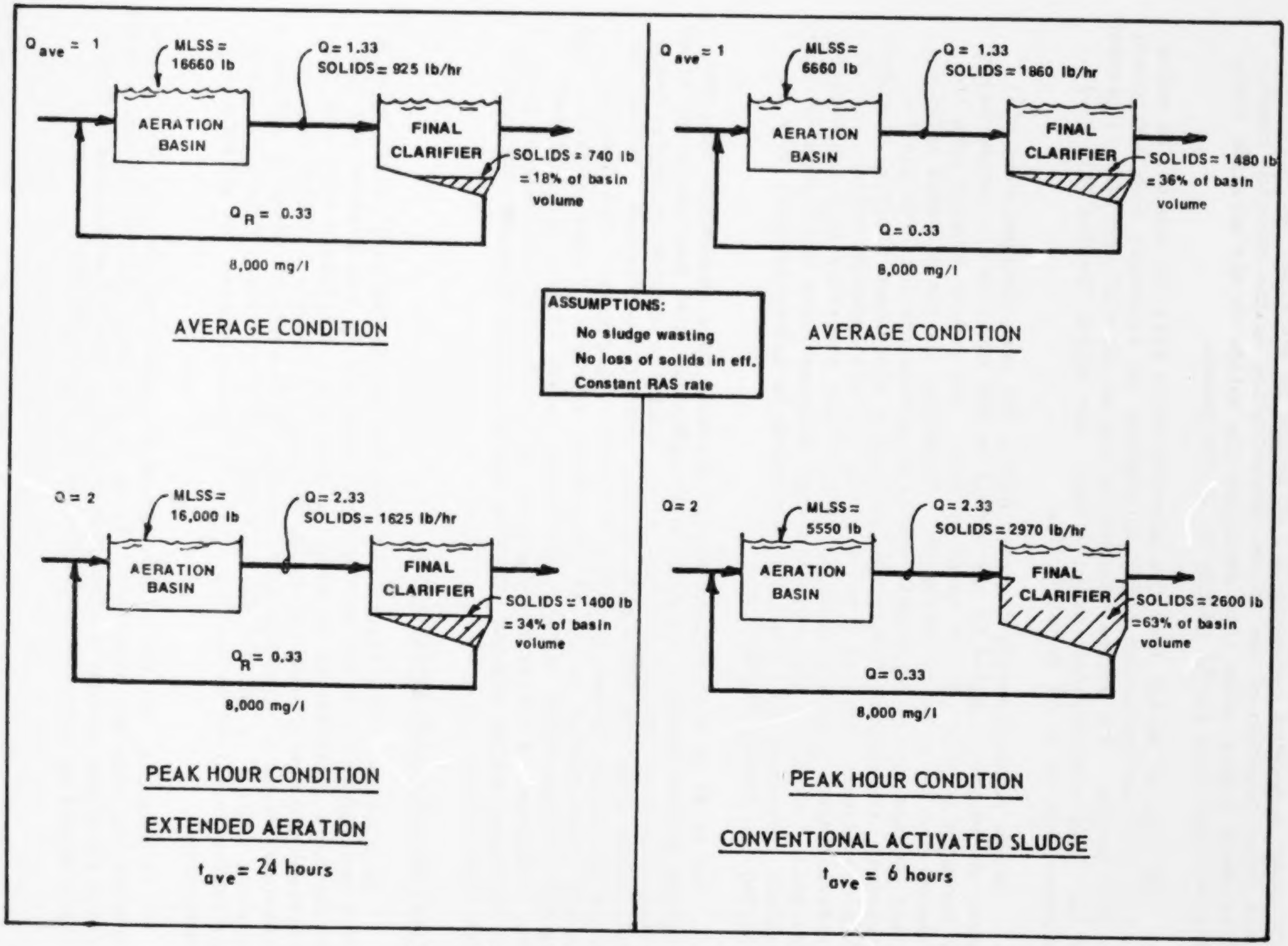

Q_{ave} = 1
MLSS = 16660 lb
Q = 1.33
SOLIDS = 925 lb/hr
AERATION BASIN
FINAL CLARIFIER
SOLIDS = 740 lb = 18% of basin volume
Q_R = 0.33
8,000 mg/l
AVERAGE CONDITION
Q_{ave} = 1
MLSS = 6660 lb
Q = 1.33
SOLIDS = 1860 lb/hr
AERATION BASIN
FINAL CLARIFIER
SOLIDS = 1480 lb = 36% of basin volume
Q = 0.33
8,000 mg/l
AVERAGE CONDITION
ASSUMPTIONS:
No sludge wasting
No loss of solids in eff.
Constant RAS rate
Q = 2
MLSS = 16,000 lb
Q = 2.33
SOLIDS = 1625 lb/hr
AERATION BASIN
FINAL CLARIFIER
SOLIDS = 1400 lb = 34% of basin volume
Q_R = 0.33
8,000 mg/l
PEAK HOUR CONDITION
EXTENDED AERATION
t_{ave} = 24 hours
Q = 2
MLSS = 5550 lb
Q = 2.33
SOLIDS = 2970 lb/hr
AERATION BASIN
FINAL CLARIFIER
SOLIDS = 2600 lb =63% of basin volume
Q = 0.33
8,000 mg/l
PEAK HOUR CONDITION
CONVENTIONAL ACTIVATED SLUDGE
t_{ave} = 6 hours

Figure 15. Effect of hydraulic variations.

will cause a greater concentration of solids in the final sedimentation basin and a dilution of the solids concentration in the aeration basin. After about 1 hour under this condition the solids in the aeration basin will decrease from 16,660 pounds to 16,000 pounds.

The loss of solids from the aeration basin will, of course, be added to the final sedimentation basin, increasing the inventory from 740 pounds to 1400 pounds. The volume occupied by the solids will approach 34 percent of the final sedimentation basin volume. The solids loading rate will increase from 13 to 24 lb/day/sq ft.

In the conventionally designed plant, the same circumstances will cause the volume occupied by the sludge in the final sedimentation basin to increase from 36 percent to 63 percent. The solids loading rate increases from 27 to 43 lb/day/sq ft. Therefore, as the extended aeration plant remains within reasonable operating parameters for high quality treatment, the conventionally designed plant approached marginally acceptable conditions. This is a major reason for the apparent reliability of oxidation ditch plants especially if the final clarifier is conservatively designed. In effect the conventionally designed plant would require operational procedures to adjust for the change in hydraulic load, such as varying the sludge recycle rate.

The shift in solids inventory is actually more pronounced than the example depicts because the peak daily hydraulic load does not occur after establishment of equilibrium conditions under the average hydraulic load, but occurs after the night time minimum hydraulic conditions. During the night the solids inventory shifts to the aeration basin. The greater solids concentration in the aeration basin at onset of peak hydraulic load causes a higher final sedimentation basin solids influx than depicted. The management of solids inventory for the conventionally designed plant is as important during minimum flows as during maximum flows to compensate for this effect.

Final Clarification Evaluation

Many of the final clarifiers observed during the field visit program were operating within the design range outlined in this section and were producing a satisfactory effluent. Those final clarifiers that were not performing properly were generally deficient in physical design such as sidewater depth or were operating at excessive overflow rates or solids loading rates. It is felt that the application of final clarifier design parameters outlined in Section 2 will generally provide for satisfactory operation.

If possible, two clarifiers should be considered although most plants visited had only one clarifier. It is nearly impossible to service a single final clarifier without discharging partially treated wastewater. Clarifiers should be inspected and serviced at least annually.

Solids Production and Treatment

Any biological plant design must consider the quantity of sludge produced, the nature and stability of the sludge, and suitable disposal procedures. An oxidation ditch plant, operated in the extended aeration mode, has certain inherent advantages relative to sludge handling and disposal. Oxidation ditch plants can be operated at a 20 to 30 day SRT resulting in a sludge having characteristics similar to a well stabilized aerobically digested sludge. A conventional activated sludge plant operated at a 4 to 10 day SRT will produce a sludge with high residual biodegradable organic content. If placed on drying beds or on the land, it will become odorous and objectionable. Aerobic digestion of this sludge for 7 to 15 days is normally required to produce a stable product suitable for disposal on drying beds or the land. For a properly operated conventional activated sludge plant plus aerobic digestion the total SRT prior to disposal will be 15 to 20 days. In effect, the oxidation ditch extended aeration process provides sludge stabilization equivalent to conventional activated sludge plus aerobic digestion.

In most oxidation ditch plants where the extended aeration process is used (16 to 24 hours aeration detention) sludge is wasted directly to open drying beds. In a few cases sludge is wasted directly to tank trucks which spread the liquid sludge on the plant grounds or on adjacent land. During this study, no odor problems were encountered at plants using either method. The author visited plants using both methods and detected no noticeable odors or any other indication of nuisance complaints. The field inspections consistently confirmed the lack of odor problems with sludge from the extended aeration oxidation ditch plants. Several plants complained of insufficient drying bed capacity. Because of poor dewaterability especially during periods of cold or wet weather. Adequate drying bed capacity must be considered carefully in design.

Some design engineers and regulatory authorities may require additional sludge stabilization for oxidation ditch plants operated in the extended aeration mode. Part of the consideration may be the possibility of periodic poor operation of the process. Also, future flow increases may force operation of the plant at shorter SRT's to the point where additional sludge stabilization is required.

The quantity of sludge produced is related to the characteristics of the incoming wastewater solids. Normally, wastewater solids contain fractions that are inert, volatile and nonbiodegradable, and volatile and biodegradable. The inert (nonvolatile) and volatile/nonbiodegradable will accumulate in the system solids inventory in proportion to the SRT. Normally 20 to 25 percent of the raw waste suspended solids are inert. The remaining 75 to 80 percent are volatile solids with, typically, 30 to 40 percent nonbiodegradable. Therefore, about half of the incoming suspended solids are not subject to biological action and will accumulate in the mixed liquor in proportion to the SRT. The effect is illustrated by assuming the following conditions:

Raw wastewater BOD_5 = 200 mg/l

Raw wastewater suspended solids = 200 mg/l

Nonbiodegradable suspended solids = 100 mg/l (50% of raw suspended solids)

Aeration detention = 24 hours

The nonbiodegradable suspended solids in the raw sewage are 0.5 lb/lb BOD_5.

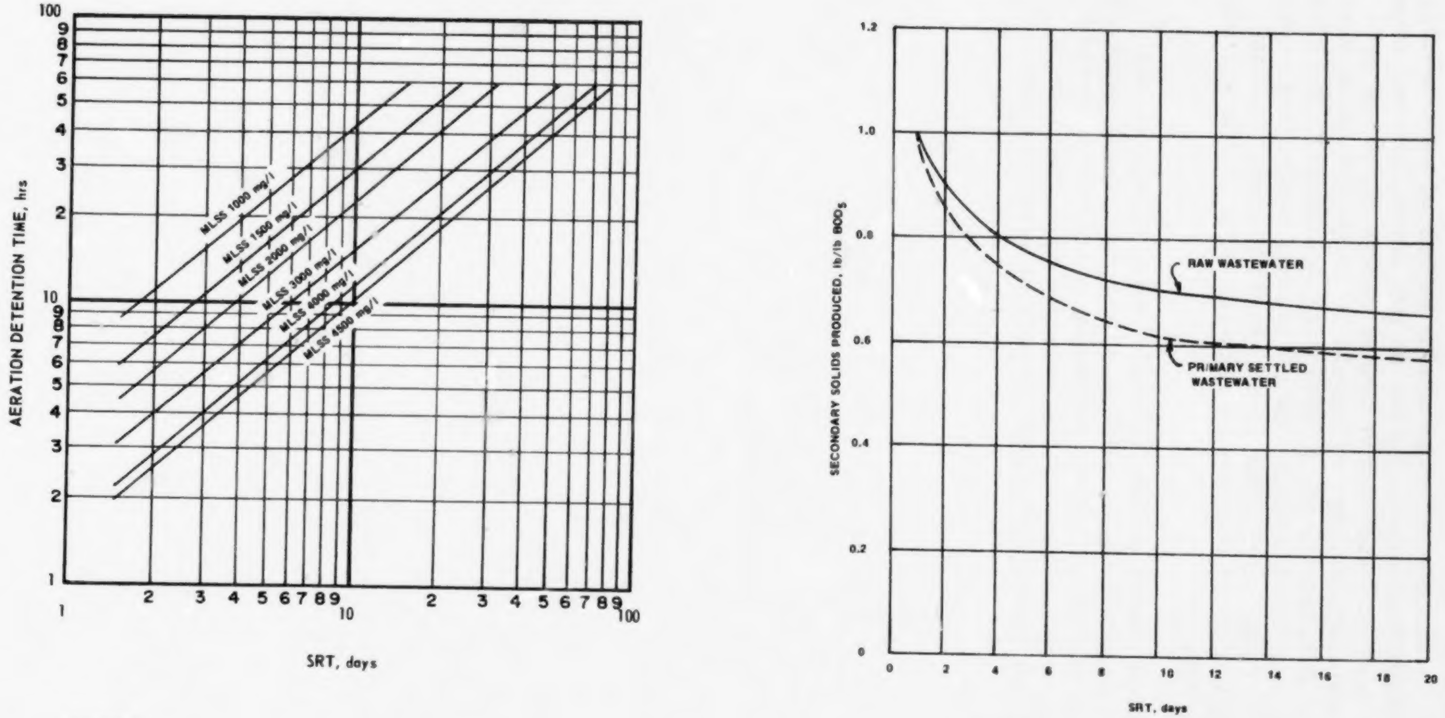
The remaining (biodegradable) suspended solids and soluble organics are synthesized into bacterial cells and loose their original identity. The 100 mg/l of biodegradable suspended solids in the assumed waste will produce approximately 0.7 lb of bacterial cells per pound of BOD_5 at low SRT. As the SRT increases the biodegradable solids are reduced until at very high SRT only about 0.15 lb of bacterial cells per lb of influent BOD_5 remain. This remaining portion of the bacterial cells is nonbiodegradable. The total solids production at low SRT would be 1.2 lb/lb BOD_5 (0.5 lb/lb BOD_5 nonbiodegradable influent plus 0.7 lb/lb BOD_5 of bacterial cells) and at high SRT would be 0.65 lb/lb BOD_5 (0.5 plus 0.15). For various SRT, based on these example conditions and the absence of primary sedimentation, the solids production has been calculated(4) and is shown in Figure 16.

The solids production is also shown with primary sedimentation preceeding aeration using the same general assumptions plus an assumed 50 percent suspended solids and 30 percent BOD_5 removal in primary sedimentation.

If the effluent suspended solids concentration averaged 25 mg/l, the remaining sludge production would normally be removed from the system by the waste activated sludge (WAS). For example, with an SRT of 20 days, the suspended solids production for the raw wastewater condition is 0.68 lb/lb BOD_5. At a final effluent solids concentration of 25 mg/l the effluent solids would represent 0.125 lb/lb of influent BOD_5. Therefore, sludge wasting would be based on 0.56 lb/lb BOD_5 applied.

The SRT, aeration detention time and MLSS are related as shown in Figure 18 for the conditions assumed. This relationship would apply to a given plant (where aeration detention time would normally be a physical constant) and also illustrates the interrelationship of these parameters for various activated sludge plants. The SRT is a direct function of MLSS and aeration detention time. If aeration detention time is a constant, the SRT and MLSS are directly related. This graphical relationship assumes steady state operation with sludge wasting rate equal to incoming solids.

Though wastewater solids are continually being synthesized, it is possible to store activated sludge in the aeration system by allowing the mixed liquor suspended solids to increase. Solids can then be wasted on



Assumptions:

Temperature = 20° C
200 mg/l BOD_5 to aeration
Solids wasted for stable operation at each condition.

Assumptions:

Temperature = 20° C
200 mg/l raw BOD_5
200 mg/l raw suspended solids
100 mg/l raw non-biodegradable suspended solids
24 hours aeration detention
Primary sedimentation – 50% suspended solids removal
30% BOD_5 removal
Calculated based on unified model (4).

Figure 18 . Biological process mixed liquor relationships and solids production.

a periodic schedule convenient with other plant operations or with sludge disposal operations. It is recommended that sludge be wasted on a regular schedule, however, to assure that sludge solids do not accumulate and cause operating difficulties. Adequate and simple sludge return should be provided. Sludge control (telescoping) valves should be large enough so they do not plug with material passing the plant screening equipment. The sludge return rate should be measured, adjustable and of minimal mechanical complexity. Generally, non-clog centrifugal pumps have proven satisfactory. The return pumps can be eliminated if sludge can be returned back to the wet well of the raw sewage lift pumps.

Some extended aeration plants do not formally waste sludge at all, however because of the nonbiodegradable content of the wastewater suspended solids it is obvious that the mixed liquor suspended solids will continue to increase. Eventually, some solids must be released from the plant either through high effluent suspended solids levels or a major plant upset.

SECTION 3

PERFORMANCE AND RELIABILITY

GENERAL

As a group, the oxidation ditch plants appeared to perform consistently well in spite of limited operation and maintenance in a number of cases. All plants appeared to have adequate aeration capacity, adequate velocity in the ditch, an acceptable mixed liquor and a lack of odor or other nuisance. In general, the operators were able to obtain good treatment results in almost all cases. Sludge handling and disposal was relatively simple and trouble free, however, in some cases operators reported difficulties in dewatering the sludges on open drying beds due to lack of proper bed area or due to slow dewatering of the sludge. The plants were easy to keep in service and would operate for long periods of time with little operation and maintenance. Many plants operated for periods of time unattended (evenings and weekends) without significant problems. No particular type of oxidation ditch plant seemed to stand out as superior or substandard; the relative performance depended on many more factors such as original design criteria (clarifier surface overflow rate for instance) and operational procedures. There are some exceptions to these comments and a few oxidation ditch plants have been removed from service because of operational problems. These cases are the exception.

Performance and reliability data for oxidation ditch plants were developed from actual plant operating records, obtained from published literature, telephone and letter contact with operating plants, visits to operating plants, EPA records, special studies, and the contractor's files. There were very few plants where complete data could be obtained for all desired parameters. In addition, reliance had to be placed on the sampling and analysis methods used by various plant personnel. It is recognized that these procedures are not consistent from plant to plant and therefore, not always directly comparable. Nevertheless, the results presented herein represent the data as obtained without modification. Inordinately high or low readings were not removed from the data during compilation because most plants experience these variations at one time or another. It is felt that these variations do occur in oxidation ditch plants as in most other activated sludge plants. In some cases a very limited number of data points was available and this is noted in the supporting tables to follow.

PERFORMANCE

Oxidation ditch plant performance was developed primarily from monthly average data. Generally, the data from a plant were not used unless several data points were available. Plant daily performance data, when available, were converted to monthly averages, which were then analyzed for each plant.

The sources of data and number of monthly average data points used are indicated in Table 5 along with some pertinent plant parameters. Where possible, the performance is calculated for both summer and winter. Winter is arbitrarily determined to be the months of November through March. The "average" performance was determined by averaging the performance of all of the individual plants. The high and low individual plant performance is shown to establish the range limits of individual plants.

Performance averages are summarized in Table 6. Performance data points as a function of plant size are shown in Figure 19. Both BOD_5 and suspended solids removal appear to be relatively independent of plant capacity. There is some indication for BOD_5 removal that small and large plants perform slightly better than mid-sized plants, but there are few data points for plants over 1 mgd in size. The small plants are performing well for BOD_5 removal in comparison to other size plants.

RELIABILITY

The reliability curves were developed from actual daily plant data. Averaged data was not used because averaging would remove the data peaks and minimums. In general, the same comments relating to quality of data apply here as with the performance data. Readings below 5 mg/l BOD_5 were considered to be 5 mg/l because of analytical limitations inherent in the 0 to 5 mg/l range. This assumption has a minimal effect on the reliability curves and would affect only the low end of the curves. Reliability curves were plotted for summer, winter, and total year for the average of all plant data. In addition, the best and worst plant reliability data is plotted. The sources of data used for the reliability curves are shown in Table 7. The reliability is shown in Figures 20 through 23.

There is a slight trend of somewhat improved BOD_5 and suspended solids reliability in the summer over winter conditions. There is also a significant variation in reliability between the best and worst plants. The reliability of the best plants is outstanding and approaches that of activated sludge plus effluent filtration as shown in Figure 30. Even the worst plants are able to meet a 20-20 standard 50 percent of the time and a 30-30 standard 70 to 80 percent of the time.

The COD and total nitrogen reliability is based on a limited availability of data, primarily from the Dawson, Minnesota plant (23). The Dawson plant was operated in a unique mode during the period represented by these data and, therefore, any use of these data should be preceded by a study of the referenced report. Various considerations relating to nitrification and nitrogen removal are covered in Section 4.

TABLE 5. SOURCES OF PERFORMANCE DATA

No.	Facility and type		Design capacity, mgd	Actual flow, mgd			Number of monthly average data points used in performance calculations															Data Source
				Winter	Summer	Annual	Inf. BOD$_5$	Eff. BOD$_5$	Inf. COD	Eff. COD	Inf. SS	Eff. SS	Inf. F. Col.	Eff. F. Col.	Inf. P	Eff. P	Inf. TKN	Eff. TKN	Inf. Ammonia	Eff. Ammonia	Eff. Nitrate	
1	Bolivar, MO	(P)	1.34	N/A	N/A	0.79	4	4			4	4							4	4		EPA Region VII
2	Battle Creek, NE	(1)	0.21	N/A	N/A	0.21	4	4			4	4							4	4		EPA Region VII
3	Laurel, NB	(L)	0.16	N/A	N/A	0.08	4	4			4	4							4	4		EPA Region VII
4	Nixa, MO	(L)	0.41	N/A	N/A	0.16	4	4			4	4							4	4		EPA Region VII
5	Salem, MO	(L)	0.74	N/A	N/A	0.50	4	4			4	4							4	4		EPA Region VII
6	Seymour, MO	(L)	0.26	N/A	N/A	0.23	4	4			4	4							4	4		EPA Region VII
	Florence, S.C.																					
7	Airport	(L)	0.12			0.12	3	3			3	3		3								Plant Visit
8	IRBEN St.	(L)	0.11			0.11	9	9														Plant Visit
9	Kershaw, S.C.	(E)	0.50	0.18	0.22	0.20	12	12														Plant Visit
	Dillon, S.C.																					
10	Pee Dee	(E)	0.80	0.96	0.90	0.92	27	27														Plant Visit
11	Maple Swamp	(E)	0.36	0.42	0.35	0.36	27	27														Plant Visit
	Clayton County, GA																					
12	Jackson	(L)	1.00	0.44	0.45	0.44	12	12			12	12		12								Plant Visit
13	N.E.	(P)	1.00	0.69	0.53	0.59	12	12			12	12		12								Plant Visit
14	Huntsville, TX	(E)	0.80	0.54	0.63	0.60	12	12			14	14		14								Plant Visit
15	Paris, TX	(E)	4.62	N/A	N/A	3.90	18	18			18	18							1	1	1	Plant Visit
16	Daingerfield, TX	(L)	0.70	0.35	0.25	0.31	14	14				14										Plant Visit
17	Whitewright, TX	(L)	0.25	N/A	N/A	0.12								28*								Plant Visit
18	Dunsmuir, CA	(L)	0.50	0.39	0.39	0.39	9	9			5	5		9								Letter
19	Saanich, B.C.	(L)	0.36	N/A	N/A	0.36	26*	26*			26*	26*										Lakeside
20	Berthoud, CO	(L)	1.10	0.44	0.66	0.56	12	12			12	12		8								Lakeside
21	8 New England Plants		0.38	N/A	N/A	0.22	8	8			8	8		8								EPA Region I
22	Greenville, OH	(L)	3.00	N/A	N/A	3.00	6	6														Lakeside
23	W. Liberty, OH	(L)	0.20	0.26	0.15	0.20	12	12			12	12										Lakeside
24	Lucasville, OH	(L)	0.63	N/A	N/A	0.20	12	12			12	12										Lakeside
25	Brookston, IN	(L)	0.20	0.22	0.19	0.22	12	12			12	12										Lakeside
26	Glenwood, MN	(L)	0.44	0.39	0.53	0.48	24	24	24	24	24	24			24	24	24	24			24	Report (15,16)
27	Somerset, OH	(L)	0.21	0.17	0.06	0.12													10	10	10	Lakeside
28	Dawson, MN	(Cherne)	0.24	-	-	0.17			1	1							8	8	1	1		Report (23)
29	Wymore, NE	(P)	0.40	0.14	0.18	0.16													12	12	12	Letter
	Average		0.73			0.54																

(L) Lakeside
(P) Passavant
(E) Envirex
* Grab samples - not monthly averages - these were averaged by month.
N/A Data not available

TABLE 6. OXIDATION DITCH PLANT PERFORMANCE SUMMARY

	EFFLUENT, mg/l			REMOVAL, %		
	Winter	Summer	Total	Winter	Summer	Total
BOD$_5$						
HIGH PLANT	55	34	41	87	86	87
AVERAGE	15.2	1.2	12.3	92	94	93
LOW PLANT	1.9	1.0	1.5	99	99	99
SUSPENDED SOLIDS						
HIGH PLANT	26.6	19.4	22.4	81	82	82
AVERAGE	13.6	9.3	10.5	93	94	94
LOW PLANT	3.1	1.9	2.4	98	98	98
COD						
AVERAGE	46	47	47	79	79	80
FECAL COLIFORM, (per 100 ml)						
HIGH PLANT	-	-	208	-	-	-
AVERAGE	41	39	40	-	-	-
LOW PLANT	-	-	6.2	-	-	-
TOTAL PHOSPHORUS						
	-	-	-	-	-	-
AVERAGE	5.8	4.9	5.3	16	21	19
	-	-	-	-	-	-
TOTAL NITROGEN (TKN)						
HIGH PLANT	10	6.6	8.1	55	61	57
LOW PLANT	-	-	3.0	-	-	72
AMMONIA NITROGEN						
HIGH PLANT	7.2	1.03	3.3	81	97	91
AVERAGE	3.8	0.6	1.8	90	98	95
LOW PLANT	0.4	0.1	0.3	98	99	99
NITRATE NITROGEN						
HIGH PLANT	29	33	31	-	-	-
AVERAGE	12.5	15.1	14.1	-	-	-
LOW PLANT	5.6	4.5	5.0	-	-	-

TABLE 7. SOURCES OF RELIABILITY DATA AND PLANT CHARACTERISTICS

No.	Facility and type	Design capacity, mgd	Actual flow, mgd Winter	Actual flow, mgd Summer	Actual flow, mgd Annual	Average infl. BOD, mg/1	Average aeration detention, hrs	Clarifier overflow, gpd/sq ft Actual	Clarifier overflow, gpd/sq ft Design	Data used in reliability curves BOD_5	Data used in reliability curves TSS	Data used in reliability curves COD	Data used in reliability curves Nitrogen	Reference
	Dillon, S.C.													
1	Pee Dee (E)	0.8	0.96	0.90	0.92	129	21	580	503	X				Plant Visit
2	Maple Swamp (E)	0.36	0.42	0.35	0.36	132	24	509	509	X				Plant Visit
	Clayton County, GA													
3	NE (P)	1.0	0.69	0.53	0.59	107	46	392	746	X	X			Plant Visit
4	Jackson (L)	1.0	0.44	0.45	0.44	150	54	187	421	X	X			Plant Visit
5	Huntsville, TX (E)	0.8	0.54	0.63	0.60	137	32	209	283	X	X			Plant Visit
6	Paris, TX (E)	4.62	N/A	N/A	3.9	197	18	388	460	X	X			Plant Visit
7	Daingerfield, TX (L)	0.70	0.35	0.25	0.31	234	54	186	357	X	X			Plant Visit
8	Dunsmuir, CA (L)	0.5	0.39	0.39	0.39	154	28	402	530	X	X			Letter
9	Brookston, IN (L)	0.2	0.22	0.19	0.22	142	22	475	442	X	X			Lakeside
10	Eureka, IL (E)	0.4	0.588	0.454	0.51	124	N/A	N/A	N/A	X	X			Letter
11	Wymore, NE (P)	0.4	0.14	0.18	0.16	235	50	228	556	X	X			Letter
12	Dawson, MN (Cherne)	0.24	-	-	0.17	155	34	378	533	X	X	X	X	Report (23)
	Average	0.92			0.71									

(L) Lakeside

(P) Passavant

(E) Envirex

N/A - Data not available

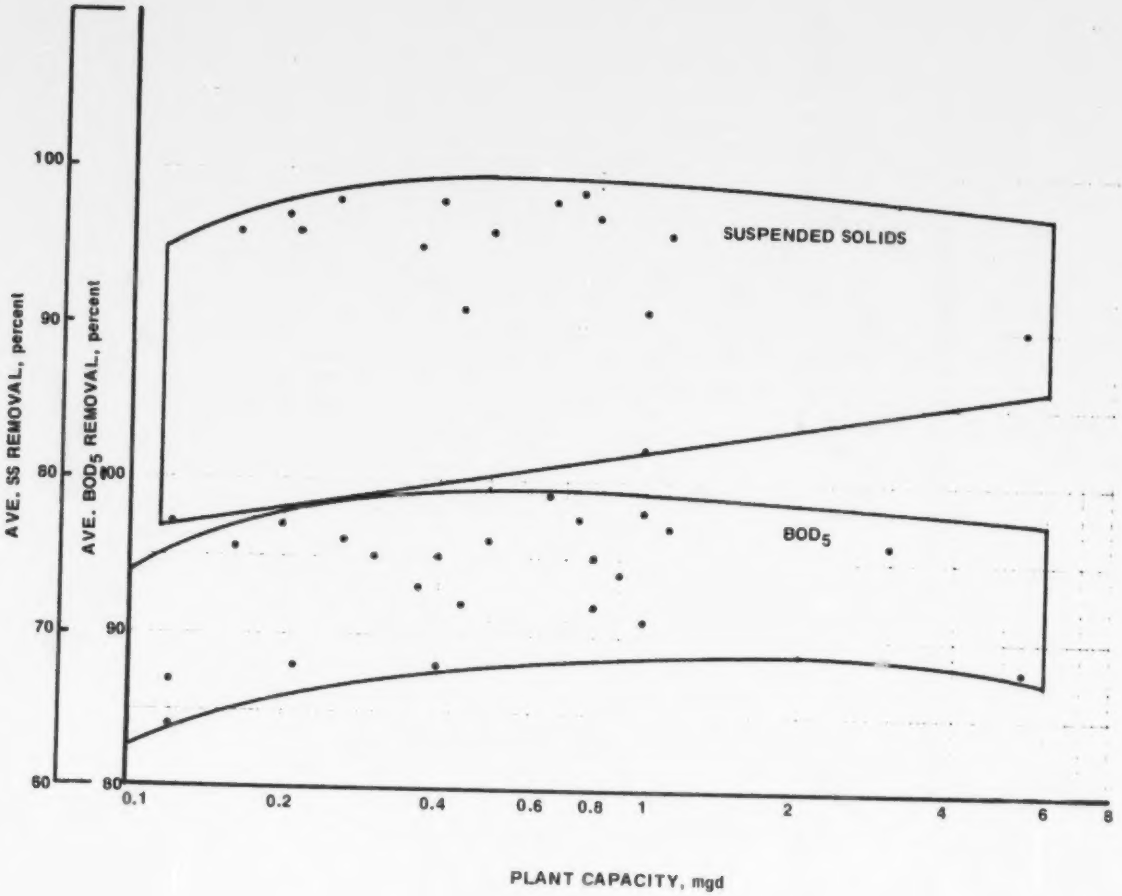


Figure 19. BOD$_5$ and TSS removal performance.

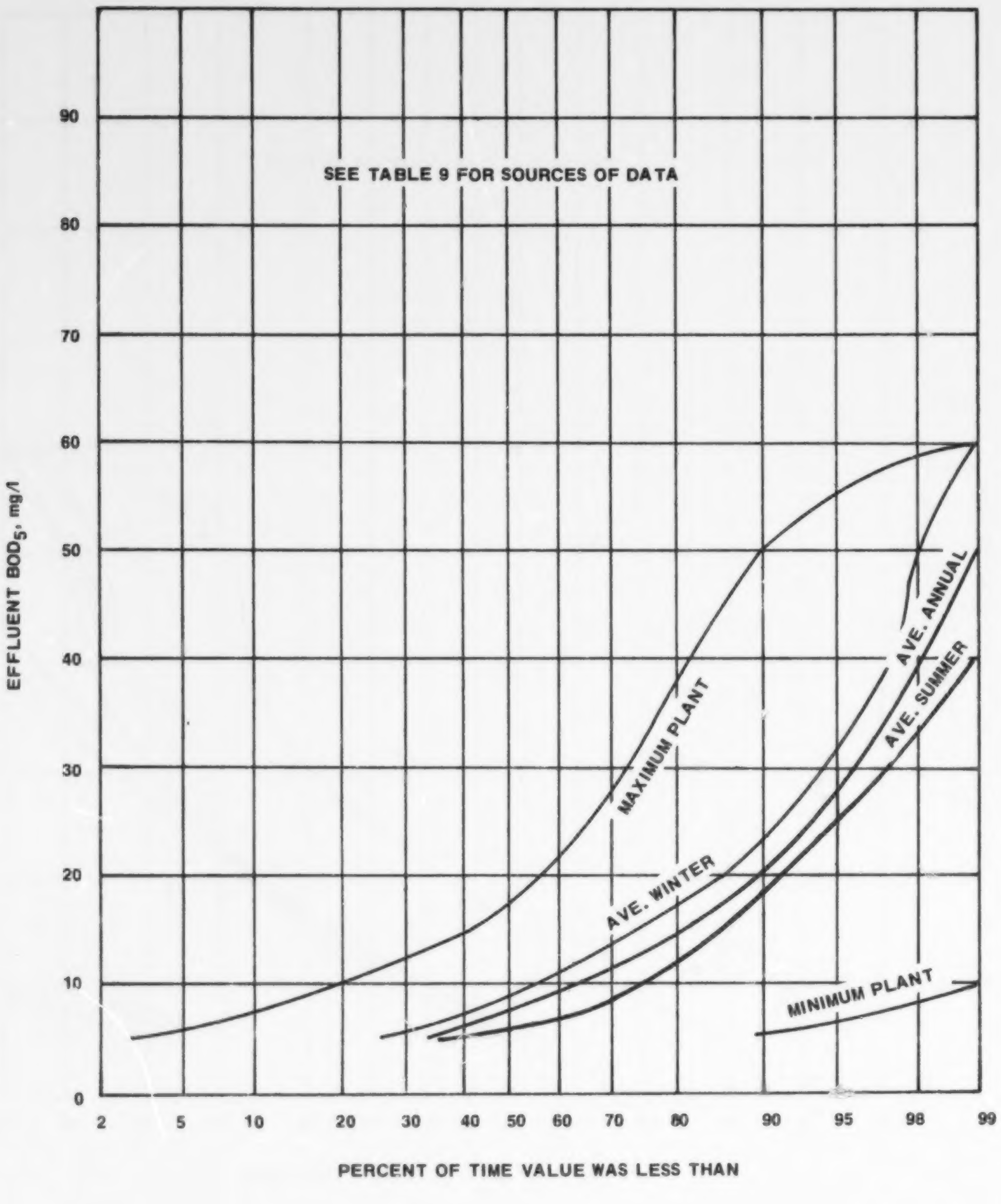


Figure 20. Oxidation ditch plant BOD$_5$ reliability.

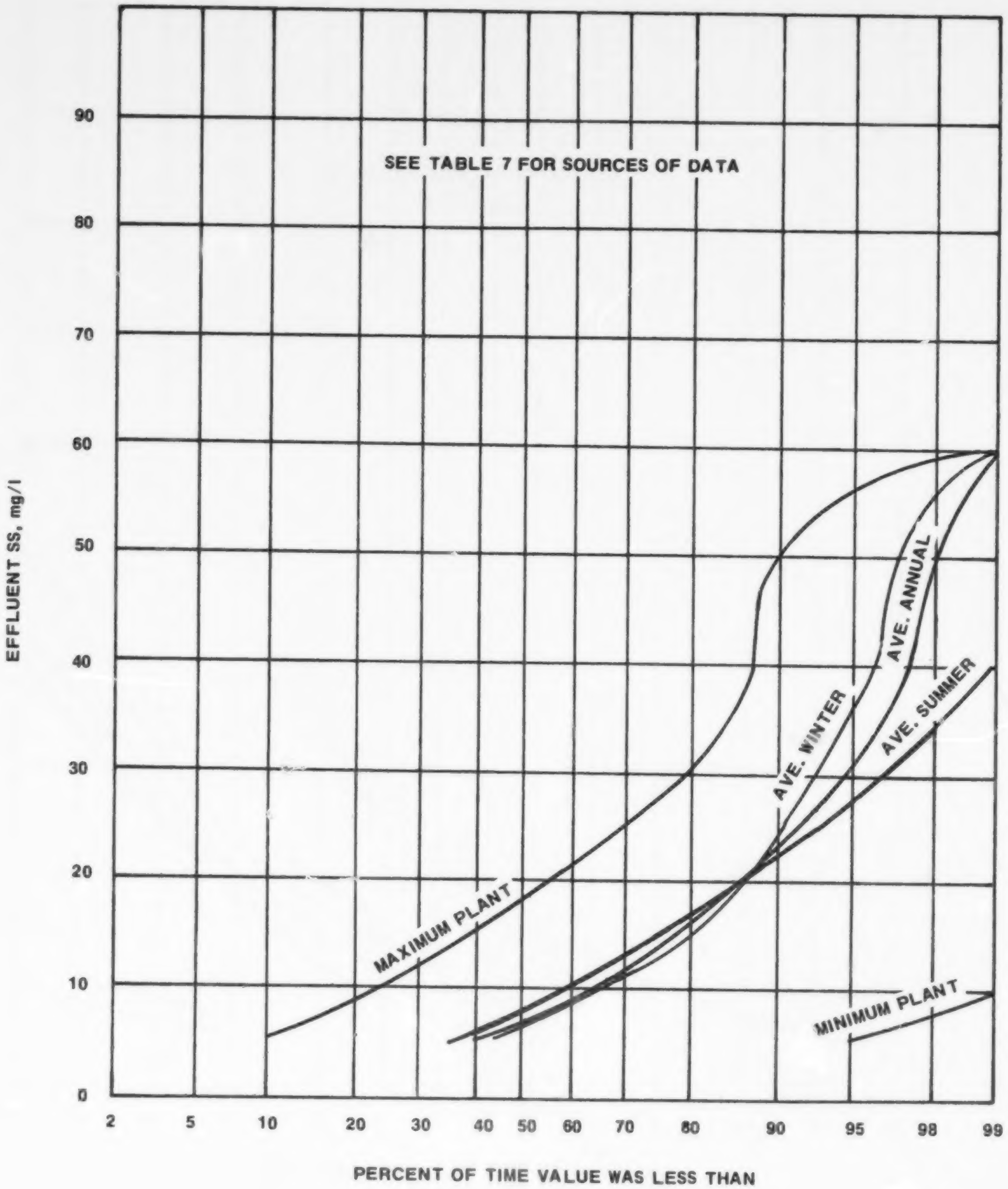


Figure 21. Oxidation ditch plant suspended solids reliability.

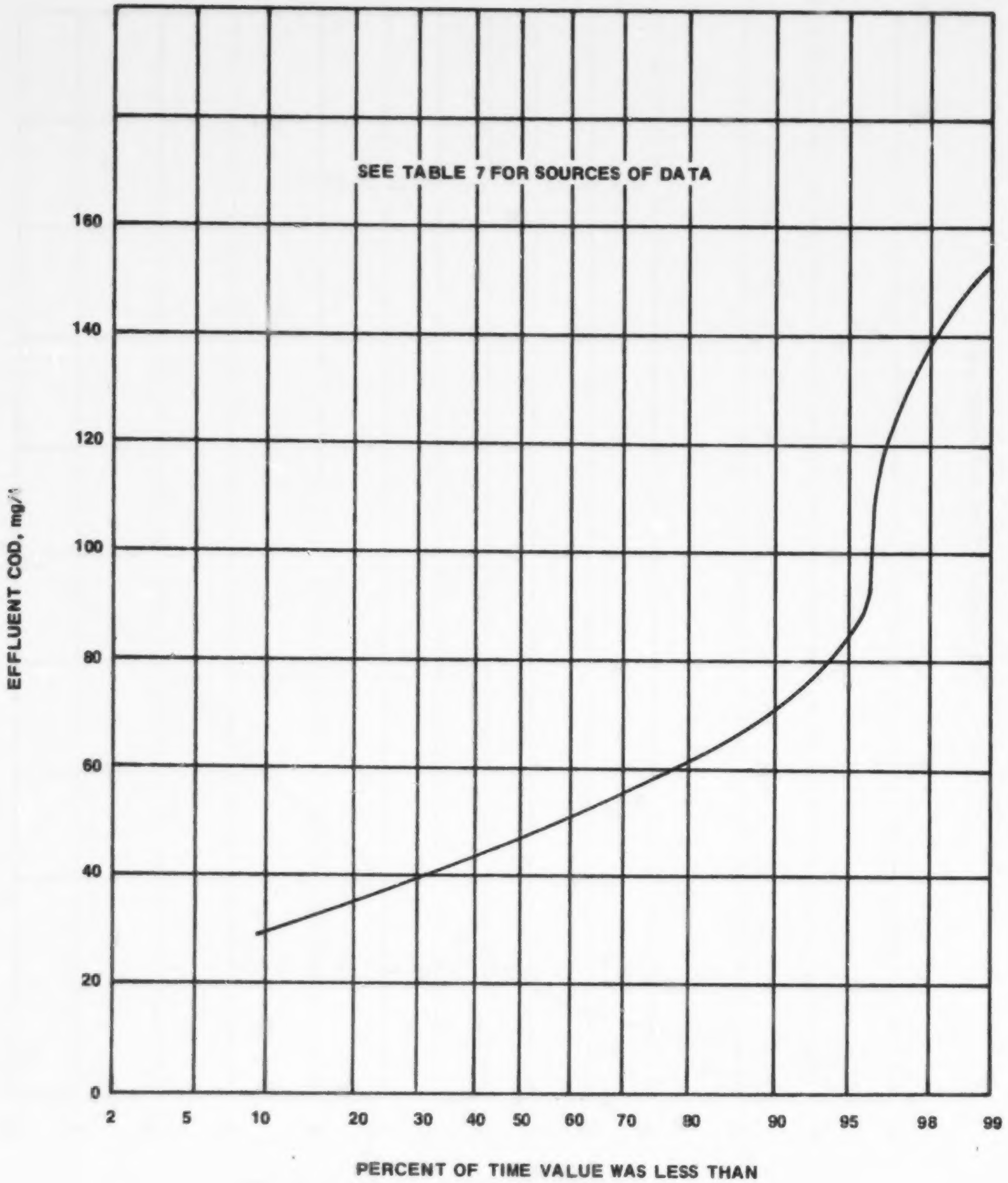


Figure 22. Oxidation ditch plant COD reliability.

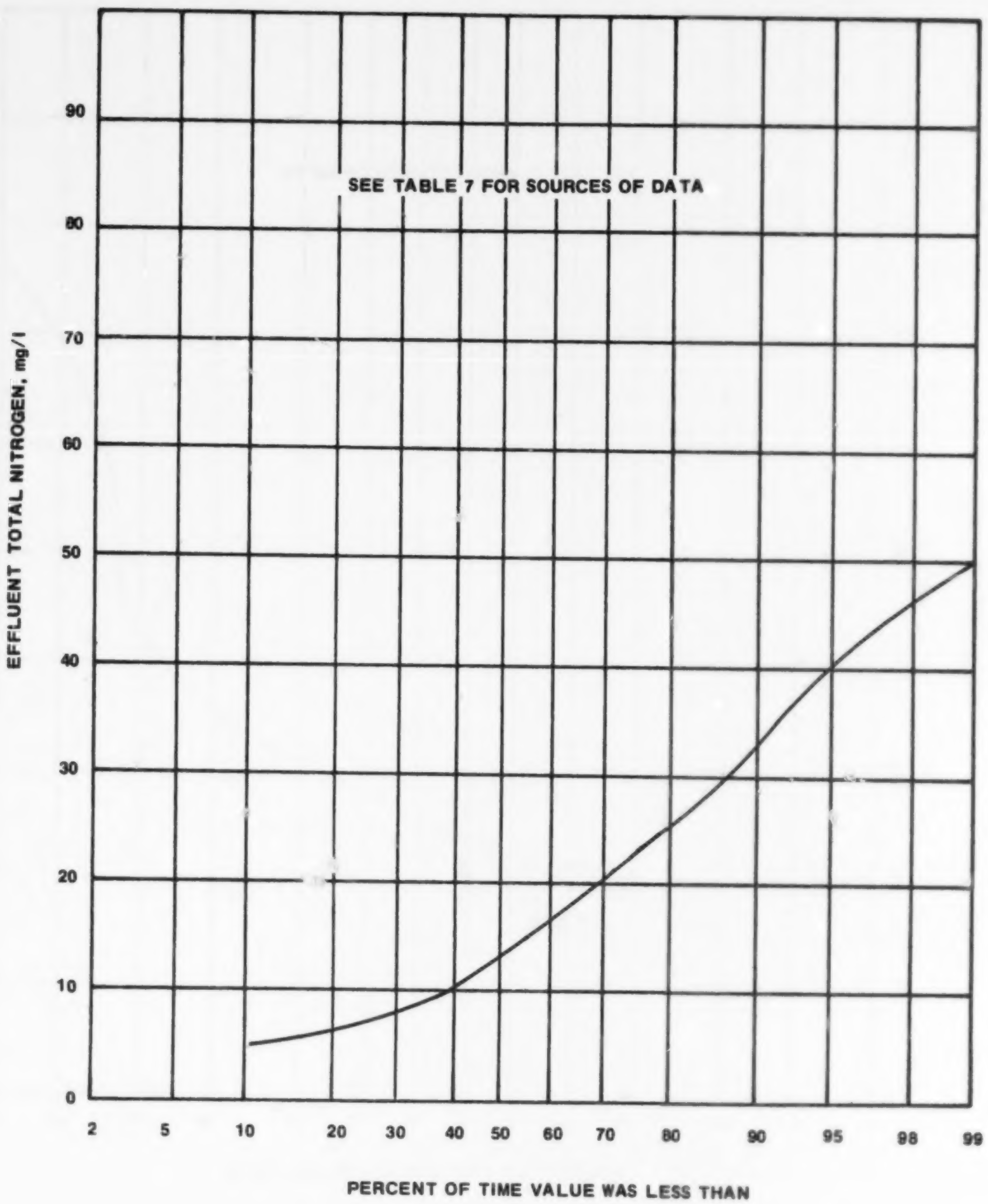


Figure 23. Oxidation ditch plant total Nitrogen reliability.

Typical dissolved oxygen and mixed liquor data are shown in Table 8. These data indicate that adequate dissolved oxygen levels are being maintained by the aeration equipment, however a number of the plants are operating below design flows as shown in Table 5. The low dissolved oxygen reported for the Dawson plant was intentional because of the special mode of operation. The mixed liquor suspended solids data are typical for activated sludge plants. At the time of data collection two of the plants were being operated at abnormally high MLSS, but operation was satisfactory in both cases.

OXYGEN UPTAKE RATE

It was nearly impossible to develop meaningful oxygen uptake rate information from data available from the plant visits. Detailed on site uptake rates were measured at Berthoud, Colorado under EPA Contract 68-03-2224(21). These tests indicate a mixed liquor uptake rate of 12 mg/l/hour or a total of 1,375 lb/day for the plant at an average flow of 0.69 mgd during the measurement period. Average BOD_5 load to the plant during the period of measurement was 816 lb/day which indicates a total rate of 1.68 lb O_2/lb BOD_5. The Berthoud Study indicates the following estimated distribution of the oxygen demand.

BOD Requirement - 56% of total

Nitrification Demand - 33% of total

Endogeneous Respiration - 11% of total

The rate without nitrification would be 1.12 lb O_2/lb BOD_5 or a sludge age of about 16 days according to Figure 13. The calculated sludge age based on average plant data during the measurement period was 21 days.

Typically, for normal domestic sewage the oxidation ditch equipment manufacturers recommend a design oxygen input of 2.35 to 2.5 lb O_2/lb BOD_5.

The Lakeside Cage aerator used at Berthoud, operating at 66 rpm with 9 inches submergence provides approximately 2.06 lb O_2/hr/lineal foot of aerator or 2,175 lb O_2/day for the 44 lineal feet of aerator. This is approximately 2.66 lb O_2/lb BOD_5 and appears to provide a margin of safety over the on site measured plant uptake rate of 1.68 lb O_2/lb BOD_5.

MIXED LIQUOR CHARACTERISTICS

The settling characteristics of the mixed liquor in oxidation ditch plants is good in most cases. In some cases, poor settling has been observed and it is felt that it may be due to over aeration or filamentous growths. Data from the Clayton County Water Authority, Georgia illustrates two typical cases; one with average MLSS and the other with

TABLE 8. OXIDATION DITCH PLANT DISSOLVED OXYGEN AND MIXED LIQUOR SUSPENDED SOLIDS.

Facility and type		Average dissolved oxygen		Average MLSS, mg/l	Reference
		mg/l	Location		
Wymore, Neb.	(P)	5.2	Final eff.	3,429	Letter
Eureka, Ill.	(E)	3.3	Final eff.	2,272	Letter
Whitewright, Tx.	(L)	2.8	Final eff.	N/A	Plant Visit
Ft. Stockton, Tx.	(L)	2.5	Final eff.	N/A	Letter
Greenville, Ohio	(L)	1.5	Ditch$^{(*)}$	N/A	Lakeside
West Liberty, Ohio	(L)	5.6	Final eff.	N/A	Lakeside
Lucasville, Ohio	(L)	5.8	Final eff.	N/A	Lakeside
Dunsmuir, Ca.	(L)	3.5	Ditch$^{(*)}$	N/A	Letter
Clayton County, Ga.					
N.E. Plant	(P)	2.6	Final eff.	3,627	Plant Visit
Jackson Plant	(L)	5.8	Ditch$^{(*)}$	8,216	Plant Visit
Baldwin, Ga.	(P)	6.6	Ditch$^{(*)}$	N/A	Plant Visit
Kershaw, S.C.	(E)	5.0	Final eff.	N/A	Plant Visit
Huntsville, Tx.	(E)	N/A		8,510	Plant Visit
Dawson, Minn.	(Cherne)	0.71	Ditch$^{(**)}$	5,000	Report[23]

(L) Lakeside
(P) Passavant
(E) Envirex
N/A Data not available
(*) At point of drawoff of mixed liquor.
(**) Downstream of aerator, plant operated in special mode.

very high MLSS. Both report good effluent performance. These data are shown in Table 9. Most plants do not develop detailed mixed liquor performance data, however most plants visited during the field visit phase of this study produced a mixed liquor typical to the N.E. Plant shown in Table 9.

SOLIDS PRODUCTION

Few oxidation ditch plants maintain adequate records to calculate the solids production. There is no reason that this parameter for oxidation ditch plants should be any different from the parameter for other extended aeration plants. Typically this parameter should be approximately 0.7 lb/lb BOD$_5$ removed when the plant is operated at a long SRT. This relationship is shown in Figure 16.

The Wymore, Nebraska and Glenwood, Minnesota oxidation ditch plant records are adequate for calculation of this parameter. Wymore is a Passavant plant operating at less than one half of design flow and at a SRT of approximately 24 days. The calculated solids production for this plant is 0.67 lb/lb BOD$_5$. Glenwood(15)(16) is a Lakeside plant operating at design flow. The calculated solids production for this plant is 0.6 lb/lb BOD$_5$ and the approximate SRT is 27 days. Wymore raw BOD$_5$ and suspended solids are above the assumed 200 mg/l and Glenwood raw BOD$_5$ and suspended solids are about 100 mg/l each. These calculations relate to the information developed in Figure 16 and the corresponding theoretical discussion of solids production

EFFECT OF OXIDATION DITCH CONFIGURATIONS ON PROCESS PERFORMANCE

There are two basic oxidation ditch plant configurations presently in use in the United States; the single channel and the multiple concentric channel. Performance data are available for both configurations. A comparison of suspended solids and BOD performance data from a limited number of plants of each configuration shows no significant differences.

TABLE 9. TYPICAL MIXED LIQUOR SETTLEABILITY

N.E. PLANT, CLAYTON COUNTY WATER AUTHORITY, GEORGIA

Month	Number of tests	MLSS, mg/l	30 min settleable solids, mg/l	Basin D.O., mg/l	SVI	Effluent quality, mg/l	
						SS	BOD_5
Oct 1975	5	2540	637	3.1	252	11	6
Nov 1975	8	2940	66	2.1	211	11	7
Dec 1975	6	3667	580	4.3	163	15	6
Jan 1976	7	3945	510	4.4	135	11	8
Feb 1976	8	3800	368	3.9	99	29	13
Mar 1976	9	2576	172	2.7	64	67	28
Apr 1976	8	3722	369	2.8	94	22	13
May 1976	8	4512	392	2.6	89	18	8
June 1976	9	3933	309	2.9	77	22	6
July 1976	9	3756	271	3.1	69	29	6
Aug 1976	7	3964	273	3.2	69	15	4
Sept 1976	7	4172	290	3.1	69	19	5
Average	91	3294	353	3.2	116	22	9
R.L. JACKSON PLANT, CLAYTON COUNTY WATER AUTHORITY, GEORGIA							
Oct 1975	4	7375	980	5.6	133	24	8
Nov 1975	4	7210	980	6.3	127	13	4
Dec 1975	3	8010	980	6.7	122	8	5
Jan 1976	4	9215	987	8.3	107	9	3
Feb 1976	4	9238	980	5.8	106	12	3
Mar 1976	2	7650	985	5.0	129	11	3
Apr 1976	4	9587	980	4.1	102	8	3
May 1976	3	8907	990	4.7	111	8	2
June 1976	4	8312	987	5.6	119	10	2
July 1976	5	7442	983	5.7	140	19	3
Aug 1976	3	7987	980	5.7	123	17	3
Sept 1976	5	7658	972	6.0	127	17	4
Average	41	7658	982	9	121	13	4

SECTION 4

NITRIFICATION AND NITROGEN REMOVAL

GENERAL

There is documented evidence that some degree of biological nitrification and nitrogen removal can be accomplished in oxidation ditch plants. Pasveer(24) has claimed 90 percent removal of nitrogen and others report nitrogen removals up to the 80 percent level reported by Mulbarger(25) for a three-stage system. Reports from other oxidation ditch plants indicate varying degrees of nitrification and nitrogen removal as shown in Table 10.

Nitrification

The oxidation ditch is capable of substantial to complete nitrification without design modifications. This is assuming that it is designed for extended aeration operation. Only operational modifications are required to assure maintenance of optimum conditions for nitrification. The factors which affect nitrification are:

1. Aeration detention time
2. Specific concentration of ammonia related to the MLVSS
3. Mean cell residence time
4. Temperature
5. pH
6. D.O. Level
7. BOD concentration
8. Toxic materials

If a high MLVSS level is maintained in the ditch then only the foregoing factors 4, 5, 6, 7, and 8 must be considered in operating for high degrees of nitrification.

As long as pH is in a nominal range (approximately 7.5 to 9.5) it should have little effect. During cold weather it may be desireable to maintain pH near the theoretically optimal level of 8.0 to 9.0 and some

TABLE 10. AMMONIA AND TOTAL NITROGEN PERFORMANCE DATA

Plant location	Design flow, mgd	Ammonia removal, %	Nitrogen removal, %	Source
Dawson, MN	0.24	99	>80(+), 51(*)	Study(23)
Vienna-Blumenthal(19) (operated - 2 stage)	22.8	79	85	EPA-Design Manual, Nitrogen Control
Lichtenvoorde Holland	3.2	96+	60(**)	Envirobic Systems
Winterswijk Holland	6.2	95	40(**)	Envirobic Systems
Calhoun, GA	7.0	91	47	EPA Region 4
Seymour, MO	0.26	98.4	30	EPA Region 7
Salem, MO	0.74	97	65	EPA Region 7
Nixa, MO	0.41	98.4	43	EPA Region 7
Bolivar, NE	1.34	98	20	EPA Region 7
Laurel, NE	0.16	14.1	-	EPA Region 7
Battle Creek, NE	0.21	95	65	EPA Region 7
Somerset, OH	0.21	99+	-	Lakeside Mfg. Study(15)(16)
Glenwood, MN	0.44	-	57	
Wymore, NE	0.40	91	-	Plant Study(21)
Berthoud, CO	1.1	80	70	

(*) Average over study period including non-optimum periods

(**) Not designed for N removal

(+) During optimum operation

chemical addition may be required to accomplish this depending on ammonia levels and alkalinity of the wastewater. The first choice would be to increase the mixed liquor suspended solids if possible.

The effect of factors 6 and 7 are not critical as long as adequate aeration capacity is available to maintain a dissolved oxygen level greater than 1 mg/l. Variation of influent BOD concentration of typical municipal wastewater should have no effect on nitrification in an oxidation ditch plant provided adequate aeration capacity is available.

Certain toxic materials will affect nitrification as with any biological process. Toxic materials in normal domestic sewage will normally be well below the toxic concentration. An indication of the effect of heavy metals and organics is shown in the EPA Nitrogen Manual(27), Section 3.2.9.

Nitrification rates are significantly inhibited at low temperatures. With long hydraulic detention times, high mean cell residence times, and the maintenance of a reasonable MLVSS level in the oxidation ditch high nitrification can be achieved even in cold weather. These relationships are shown in the EPA Nitrogen Manual(27) and need not be repeated in this study. In addition, the pH can be maintained in the range of 8.0 to 9.0 during cold weather as a further step in optimizing cold weather nitrification although this would normally be done only when absolutely necessary. There is evidence from many oxidation ditch plants as shown in Table 10 that high levels of nitrification can be obtained year around. The Wymore, Nebraska oxidation ditch plant averaged 91 percent reduction in ammonia nitrogen during the year 1975-1976. This average was 96.7 percent for the months of March through December and dropped to 56 and 73 percent for the months of January and February respectively. Nitrification in the oxidation ditch plant is a biological process and is subject to upset from time to time. Care and attention in operating the plant should minimize the periods of upset. Significant populations of nitrifying organisms develop slowly, therefore, more time is required to establish or reestablish a population of these organisms compared to typical activated sludge organisms. These establishment times would be greater when liquid temperatures are cold. This is shown in Figure 3-7 of the EPA Nitrification Manual(27).

Nitrogen Removal

A limited amount of work has been done with single stage biological nitrification-denitrification in oxidation ditch plants. Nitrification has already been discussed and has been demonstrated to occur in standard extended aeration oxidation ditch plants and is a function of operational parameters more than design modifications to standard plants. It is expected that substantial nitrification occurs in most oxidation ditch plants.

The oxygen profile around a single channel continuous oxidation ditch can vary considerably depending on the aerator locations, ditch length, point of introduction of raw sewage and return sludge, and the submergence, size, and speed of the aerators (oxygen transfer rate). The vertical oxygen profile within an oxidation ditch can also vary considerably especially in deep ditches. Theoretically, it is possible to control the dissolved oxygen level at any point in the ditch to a set point by varying the oxygen transfer rate of the aeration equipment (speed, submergence, or number in operation). In practice, this is not easy to accomplish either manually or automatically. The necessary monitoring equipment requires continual maintenance and the velocity imparted to the ditch contents may be inadequate at low aerator submergence or speed. Solutions to these problems are discussed in this section.

Because it is possible to create and maintain oxygen rich and oxygen deficient (aerobic and anoxic) zones around the ditch, it is then possible to create conditions for biological nitrification and denitrification within the same ditch. The nitrification takes place in the aerobic zones and the denitrification in the anoxic zones. The velocity in the ditch should be approximately 1.0 foot per second. In a typical ditch which may have a total channel length of 325 feet, the contents make a circuit about every 4 to 6 minutes. Thus, the mixed liquor and the biological contents are subjected to rapid alternation between aerobic and anoxic conditions. This rapid alternation enables the entire spectrum of organisms to survive(18). It is further believed, in this case, that the nitrate concentration in the ditch remains low because the alternating aerobic-anoxic cycles quickly denitrify the nitrates produced during nitrification in the aerobic zone(18). The denitrification process was probably not rate limited. If the denitrification process is rate limited (by lack of an adequate carbon source, for instance) the nitrates will be high as shown at Dawson, Minnesota(23).

A hydrogen donor is required in the anoxic zone and can be provided in part by distribution of the raw sewage into the channel near the start of the anoxic zone. The return sludge would also be introduced at this point.

This is basically the operation used at the Dawson, Minnesota oxidation ditch plant(23). This type operation produced sustained total nitrogen removals above 80 percent with controlled operation and average removals of 51 percent through the 10 to 12 month demonstration period.

Drews, et al(17) (18) reported on nitrogen removal work performed in South Africa at a four-channel Huisman type oxidation ditch plant. The channels were operated essentially in series with four different aeration configurations as shown schematically in Figure 24.

Mode 1 was conventional configuration with a positive oxygen level maintained in all parts of all channels. Sludge return rate for this mode was 50 percent of the raw flow.

Mode 2 was arranged for denitrification-nitrification-denitrification-nitrification. The first channel would denitrify any nitrates returned with the return sludge. The single aeration disc in the denitrifying channels (1 and 3) is to keep the mixed liquor moving to prevent settling.

Mode 3 was the reverse of Mode 2 and produced extremely poor results.

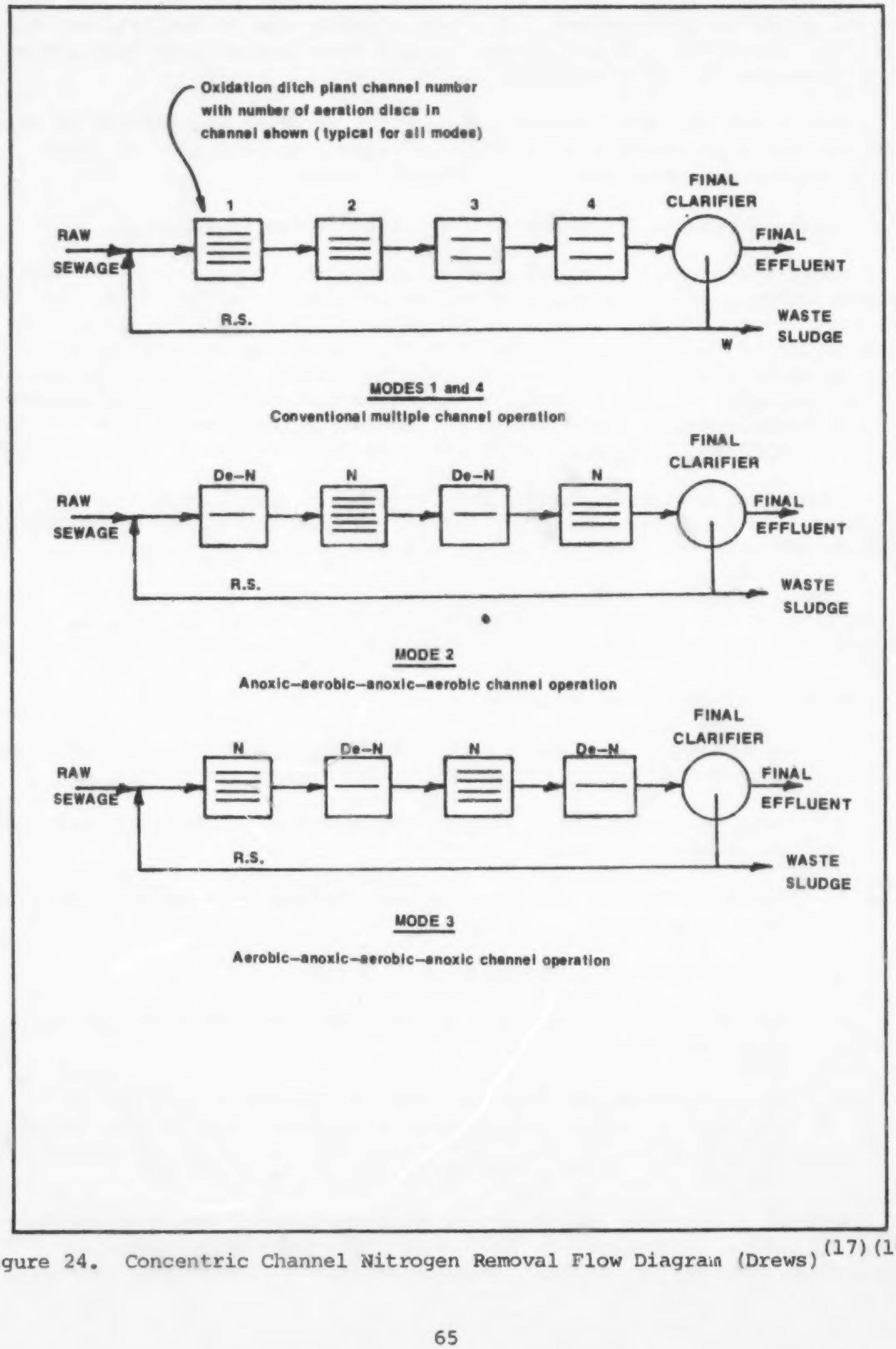

Oxidation ditch plant channel number
with number of aeration discs in
channel shown (typical for all modes)
1
2
3
4
FINAL CLARIFIER
RAW SEWAGE
FINAL EFFLUENT
R.S.
WASTE SLUDGE
W
MODES 1 and 4
Conventional multiple channel operation
De–N
N
De–N
N
FINAL CLARIFIER
RAW SEWAGE
FINAL EFFLUENT
R.S.
WASTE SLUDGE
MODE 2
Anoxic–aerobic–anoxic–aerobic channel operation
N
De–N
N
De–N
FINAL CLARIFIER
RAW SEWAGE
FINAL EFFLUENT
R.S.
WASTE SLUDGE
MODE 3
Aerobic–anoxic–aerobic–anoxic channel operation

Figure 24. Concentric Channel Nitrogen Removal Flow Diagram (Drews)(17)(18)

Sludge was continuously rising in the final clarifier producing high suspended solids in the effluent. This was probably due to denitrification in the final clarifier. This mode was dropped from further consideration and no performance data were provided in the paper.

Mode 4 was the same as Mode 1 except the aeration was limited so that each channel experienced both aerobic and anoxic zones. Each of these zones comprised approximately half of each channel.

Averaged results for Modes 1, 2 and 4 are shown in Table 11.

Ammonia and total nitrogen removal were 82 to 100 and 69 to 79 percent respectively in all three modes of operation as shown in Table 11. Only the data shown in Table 11 were published in the referenced article. The total nitrogen removal experienced in Mode 1 would not normally be expected if a positive dissolved oxygen level were maintained in each of the channels during the test. In fact, positive oxygen levels could not be maintained in the channels and at times the Mode 1 operation reverted to Mode 4. This probably explains the total nitrogen removal observed for Mode 1 operation.

Generally, a turbid effluent was produced in Mode 2 operation and a floating scum layer was always present in the final clarifier. The report did not indicate the magnitude of the effluent suspended solids.

Mode 4 produced a clear effluent, but incomplete nitrification. The efficiency of the process varied considerably over a day as might be expected when attempting to maintain aerobic and anoxic zones in each channel.

Results of the study indicate:

1. Single stage nitrogen removal of 70-90 percent could be achieved without the aid of methanol or other carbon sources.

2. When maximum nitrogen removal was obtained, a clear effluent low in COD was produced.

3. Energy costs for the aerators were reduced because of close control of aeration.

4. Skilled attention is required.

5. The oxidation ditch plant is a suitable configuration for this type operation.

6. The study reported that "in order to produce a good quality effluent at winter temperatures a somewhat lower sludge concentration will have to be maintained than at summer temperatures". The validity of this conclusion can not be confirmed.

Another operational variation for nitrogen removal was used successfully at Dawson, Minnesota(23) for a short time and has been used in actual operation at the Baldwin, Georgia oxidation ditch plant. The

TABLE 11. MULTI CHANNEL OXIDATION DITCH NITROGEN REMOVAL RESULTS (Drews)(17)(18)

Parameter	Mode 1	Mode 2	Mode 4
Wastewater Temp, °C Ave.	18.4	16	18
Return Sludge Rate	1.5:1	2:1	1.5:1 to 2:1
Detention Time, Hours	13.9	18.5	13.9 to 24.0
Number of Discs	11	10	5 to 11
MLSS, Channel 4, mg/l Ave.	3,853	4,828	3,998
Ammonia -N Removal, % Ave.	95.0	100.0	82.0
COD Removal, % Ave.	95.5	94.5	95.3
Total N Removal, % Ave.	69.0	79.3	79.0

aerator(s) are operated on an adjustable on-off cycle; such as 1 hour on and 1 hour off. In practice, the cycle could be fully adjustable and controlled by an automatic timer. The Baldwin, Georgia oxidation ditch plant is presently using this mode and is attaining very satisfactory operation with substantially decreased electrical energy, however, they do not monitor any nitrogen parameters. This mode of operation would be applicable especially to underloaded plants. The solids settled during the off time, but were resuspended when operation resumed. It was a very easy way to adjust the oxygen input to the ditch and it could also, perhaps, provide alternating aerobic and anoxic cycling. Several other oxidation ditch plants were considering this mode of operation, but only because they had excessive oxygen levels in the ditch and final clarifier and felt they could substantially reduce their electrical energy usage.

In summary, there are documented results from both single channel and multiple concentric channel oxidation ditch plants and other extended aeration plants relating to nitrification and nitrogen removal. The following design and operation parameters properly applied are capable of providing essentially complete nitrification in an oxidation ditch plant at least down to mixed liquor temperatures of 1°C or, essentially, freezing.

DESIGN PARAMETERS FOR NITRIFICATION

Channel Physical Dimensions and Features

The physical length, width, and depth dimensions do not have any significant effect on nitrification as long as proper detention time is provided. There is no evidence to suggest that the single channel or the multiple concentric channel configurations possess any unique characteristics in this regard.

For a single channel plant the raw sewage and return sludge should be introduced into the ditch immediately upstream of an aerator. The mixed liquor should be withdrawn far enough upstream of the point of introduction of raw sewage to prevent any possibility of short-circuiting raw sewage directly to the final clarifier.

In a multiple concentric channel plant the raw sewage and return sludge would be introduced into the first channel and the mixed liquor withdrawn from the last channel.

Aeration Equipment

The aerator type, sizing, or spacing is not critical to nitrification as long as adequate oxygen is provided to the mixed liquor. A positive oxygen level should be maintained in all parts of the ditch under all treatment conditions. When mixed liquor temperatures are increasing, there will be additional oxygen demand because of increased volatilization of solids due to higher temperatures. This extra demand should be met to assure good nitrification. At Dawson, Minnesota a completely nitrified effluent was maintained with as little as 0.5 mg/l dissolved oxygen immediately downstream of the aerator(23). It is suggested that the goal be a positive oxygen level in all parts of the ditch and this will be achieved for normal municipal sewage if the rotors are designed for proper channel velocity.

Auxiliary Velocity Devices

The need for auxiliary devices to increase velocity in the oxidation ditch should not arise for cases involving nitrification. The aerators will impart proper velocity unless they are undersized or the ditch is improperly designed for the aeration equipment used.

Sludge Storage, Return, and Wasting

Oxidation ditch plants designed for nitrification should be designed to operate within a range of MLSS of at least 4,000 to 6,000 mg/l. Positive sludge removal should be provided from the final clarifier along with a means for returning this sludge at a known and controllable rate to the ditch. A sludge return capacity of at least 100 percent of the average flow should be provided. Proper sludge wasting is necessary including provisions for measuring the volume of sludge wasted.

It is recommended that sludge be removed from the final clarifier continuously and at a rate adequate to maintain a positive oxygen level in the return sludge. Otherwise, denitrification may take place in the clarifier which could lead to substantial carryover of solids to the effluent.

The need for auxiliary in-plant sludge storage capacity depends on a number of factors many of which are local in nature. Adequate sludge handling facilities are required so that sludge can be wasted from the plant as dictated by biological system operation rather than by sludge processing and disposal considerations. It is possible to store solids in the aeration system by allowing the mixed liquor suspended solids to increase, but this practice can lead to operational problems.

Flow Modulation

There is nothing to indicate that flow modulation or equalization is necessary to satisfactory nitrification provided the final clarifier is properly designed for expected flow variations. In cases of anticipated excessive diurnal flow variations it may be desireable to provide flow equalization storage or a method to stabilize the flow to the final clarifier such as the method used at Dawson, Minnesota(23).

Aeration Detention

If a high degree of nitrification is required all year in a relatively cold climate, the oxidation ditch plant should be designed for a detention time of 24 hours at average flow assuming a normal domestic sewage. In warmer climates where the mixed liquor temperature is expected to remain above about 15°C, a detention time of 13 hours was shown to be adequate by Drews, et al (17) (18).

Final Clarifier Sizing and Design

The final clarifier(s) should be designed for activated sludge applications with means for continuous and relatively uniform removal of sludge. The final clarifier overflow rate should be in the range of 400 to 500 gpd/sq ft at average plant flow and 1,000 to 1,200 gpd sq ft at peak flows. Design for clarifier solids loadings should be 30 lb/day/sq ft. Side wall depths should be at least 12 feet and preferably 12 to 14 feet. Deep scum baffles and double skimmer arms will be helpful to retain any rising solids resulting from denitrification in the final clarifier. It is desireable to make piping provisions so skimmings can be returned to the aeration channel.

Instrumentation

Special instrumentation should not be necessary for proper nitrification control. Therefore, only conventional instrumentation typical to an oxidation ditch plant as follows is required.

- Raw sewage or effluent flow measurement, recording & totalizing
- Return sludge flow measurement
- Waste sludge measurement
- Chlorine feed pacing to flow
- Normal laboratory instruments and equipment including a portable D.O. instrument

This instrumentation represents the minimum necessary for proper operation and recordkeeping.

OPERATIONAL PARAMETERS FOR NITRIFICATION

Oxygen Profile

No special oxygen profile is necessary in the ditch except it is desireable to have a positive oxygen level throughout the ditch except when denitrification is required.

MLSS and MLVSS

The Dawson, Minnesota plant was able to achieve essentially complete nitrification over the MLSS range of 2,600 to 10,000 mg/l (23). Typical MLSS reported by Drews, et al (18) was 3,600 to 5,500 mg/l with essentially complete nitrification reported. Theory would indicate for a 200 mg/l BOD_5 sewage and 24 hour detention time, a MLSS of approximately 3,000 mg/l would be adequate for at least 95% nitrification down to a mixed liquor temperature of 5°C. The corresponding MLVSS will vary with plant conditions, but at Dawson, Minnesota it varied from 51 to 78 percent by weight of the MLSS. Volatile solids less than 60 percent were normally obtained when the MLSS concentration was greater than 6,000 mg/l and the liquid temperature above 20°C(23). The most satisfactory MLSS for any given plant and season will have to be determined by actual operation, but a target of at least 3,000 to 5,000 mg/l is reasonable and should be achieveable consistently.

Carbon-Nitrogen Ratio

Literature(26) indicates that carbon-nitrogen ratio is not a significant factor for plants operated at a 10 day or longer SRT.

pH

The nitrification process destroys alkalinity and this may have a depressing effect on system pH if the wastewater alkalinity is not high enough to provide the residual needed for nitrification.

The rate of nitrification is pH sensitive and this characteristic is well documented. Maximum rate of nitrification occurs at a pH of about 8.5, but high rates can be expected within the pH range of 7.5 to 9.5 at a temperature of 20°C. Normally, even in a cold climate, pH control is not necessary if the MLSS is maintained at reasonably high concentration thus providing a high MLVSS.

Sludge Settleability

If a positive oxygen level is maintained in the aeration ditch and sludge is removed from the final clarifier at a proper rate, there should be little reason to expect any effect on sludge settleability as a result of the nitrification process. Over-aeration can lead to sludge settling problems because of air bubble release in the final clarifier, but this is not particularly related to nitrification. If oxygen levels are allowed to decrease to zero or if sludge is not properly removed from the final clarifier, denitrification can occur in the final clarifier. The resulting release of nitrogen gas will cause sludge to float.

Sludge Recycle Ratio

This is a variable which is difficult to generalize. The sludge return should be continuous and flow measurement and control are desireable. Flow proportional sludge return is uncommon at small plants. Normally, sludge return flow rates are maintained at about 30 to 100 percent of the average daily flow. Very high return rates up to 200% were used by Drews, et al.(18) in South Africa but this is not common practice in the United States. The sludge return rate should be adequate to maintain good clarifier performance.

Sludge Wasting Rate

Sludge wasting should be carried out to maintain the desired MLSS inventory in the biological system. When the plant is operating under relatively stable conditions, regular sludge wasting should be carried out to maintain the stability. The mixed liquor should be checked at least daily and sludge can be wasted daily or every few days. During some periods at Dawson, Minnesota, sludge did not have to be wasted for a week or more particularly in winter(23) It is better to waste nominal amounts of sludge on a regular schedule than to make large periodic adjustments. Typical sludge production is discussed in a previous section.

Solids Retention Time

Solids retention time is very important to nitrification, however, if the aeration detention is 24 hours and the raw sewage is typical domestic (200 BOD_5), operation at a solids retention time of 16 days can be achieved at a MLVSS of 1,922 mg/l and 24 days at 2,880. These are extremely conservative, should provide excellent nitrification even in very cold climates, and should be relatively easy to achieve.

Temperature

The rate of nitrification is very much a function of liquid tempreature. In cold climates anything that can be done to conserve the liquid heat will be beneficial. Excessive aeration will tend to dissipate liquid heat and therefore should be avoided. As an example, the rate of nitrification at 5°C is about 20 percent of the rate at 20°C. However, as long as the MLSS is maintained in the recommended range, very complete nitrification can be obtained even at liquid temperatures approaching freezing.

Hydraulics

No special hydraulic considerations are required over and above those dictated by proper design of an oxidation ditch plant and sizing of the final clarifier. Placement and design of the aerator structure should consider the water spray produced by the aerator. Baffles or other protective devices should be provided so this spray does not fall on bearings and gear reducers. Walkways and other accesses should be placed so the spray will not wet these areas.

DESIGN PARAMETERS FOR NITROGEN REMOVAL

Channel Physical Dimensions and Features

The channel width, depth, and length are functions of the type of aerator to be used, its length and total detention time. The Dawson, Minnesota report(23) recommends a channel length of 600 feet (for a single channel plant) in order to increase the time during which anoxic conditions are maintained. With this channel length the total time for one "revolution" of the mixed liquor would be 10 minutes and approximately 5 minutes could be provided for the anoxic zone based on a channel velocity of 1 foot per second.

The multiple concentric channel type plant should also be considered. This configuration might offer some advantages in denitrification because, conceiveably, it should be easier to establish and maintain the aerobic and anoxic zones using separate channels with separate aeration.

For a single channel plant the raw sewage and return sludge should be introduced in the vicinity of the transition between the aerobic and anoxic zones to act as a carbon source. The best location for removal of mixed liquor is open to question. One consideration would be at the end of the anoxic zone just prior to the aerator. The disadvantage of this arrangement is that denitrification will occur in the final clarifier causing rising sludge and solids carryover in the effluent. This was confirmed by Drews, et al(18) and is comparable to Mode 3 shown in Figure 24. Drawoff of the mixed liquor downstream of the next aerator and prior to reaching the anoxic zone might provide better results. This would be comparable to Modes 2 or 4 in Figure 24. Mode 4 produced a clear effluent and a total nitrogen reduction of 79 percent. Flexibility should be provided in the physical and process plant design so that field changes can be made in the operation of the plant.

The layout of the multiple concentric channel type plant would be the same as for the nitrification case; only the configuration of the aeration discs would change.

Aeration Equipment

The oxygen transfer characteristics and operating power requirements of competing aeration devices should be varified (by actual testing if necessary) in order to select the most satisfactory equipment.

It is important to avoid oversizing the aerator and some means must be provided to allow adjustment of oxygen transfer rate over a fairly wide range. This can be accomplished by adjusting speed, submergence, number in operation, and perhaps using an on-off operating cycle.

The disc type aerator used in the multiple concentric channel type plants would seem to have additional advantages for this application because the number of discs in any channel can be changed at will by adding or deleting discs on the shaft.

Spacing of aerators aroung the channel in single channel plants should provide clearly defined aerobic and anoxic zones. Specific criteria can not be provided to define this spacing. Approximate spacing can be calculated based on expected oxygen uptake rate of the specific wastewater and then, the actual capacity of each aerator can be field adjusted as previously discussed. Use of floating aerators may be beneficial because they can be moved in the field as required.

Auxiliary Velocity Devices

Conceiveably, in nitrogen removal applications, the aeration requirements may be less than the mixing or velocity requirements. In this case, the velocity in the ditch would be inadequate and solids would settle. Auxiliary flow devices were installed in the ditch at Glenwood, Minnesota (15)(16) because the aerators could not impart adequate velocity to the ditch contents at the low aeration levels required. The auxiliary flow devices were submerged propeller-like devices driven by electric motors and mounted on floating platforms. They are designed to provide velocity to the mixed liquor without adding any oxygen; hence, the reason for complete submergence of the propellers. Similar devices may be required in other plants practicing nitrogen removal where the oxygen requirements are relatively low. Plants using brush or cage aerators may experience problems with sludge settling within the channel where aerator submergence is reduced to produce lower dissolved oxygen levels within the channel. In this case, auxiliary flow devices may be required.

Sludge Storage, Return, and Wasting

These considerations should be the same as for nitrification.

Flow Modulation

One of the major problems in nitrogen removal is providing the controlled aeration required to create the proper aerobic and anoxic zones in the channel. This should be accomplished in the simplest possible manner consistent with the plant design. Diurnal variations in either flow or loading, tend to upset the stability of the aerobic and anoxic zones. Stabilization of the plant influent conditions, with processes such as flow equalization, will ease the control problem. On the other hand, perfect oxygen control as described under Instrumentation could easily accommodate a wide range of load variations satisfactorily.

Aeration Detention

Considerations related to aeration detention time are essentially the same as described for nitrification.

Final Clarifier Sizing and Design

These considerations are essentially the same as described for nitrification.

Instrumentation

Along with the basic plant instrumentation, additional requirements are needed in nitrogen removal to create and maintain aerobic and anoxic zones in a portion of the ditch. Theoretically, a dissolved oxygen probe could be immersed at the desired start of the anoxic zone and this probe used to control a variable aeration parameter(s) such as speed and/or submergence. This is not a simple task because of the need for almost constant maintenance of the probe, the lag in the system, and the range of aeration control needed. Considering normal diurnal flow changes plus normal BOD_5 load variations, both speed and submergence of the rotor would have to be varied to obtain a 4:1 range in oxygen transfer rate. It is questionable whether a satisfactory automatic control scheme meeting these requirements is available. Perhaps this control can be approximated manually with careful operator attention. It is not known how much the anoxic zone size can vary and still achieve good nitrogen removal. The experience at Dawson, Minnesota demonstrates that the required control can be accomplished manually under carefully controlled conditions(23). A high degree of operator attention may be required to achieve consistently high removal of nitrogen at typical oxidation ditch plants. Additional full scale plant experience is needed regarding the application of automatic controls.

OPERATIONAL PARAMETERS FOR NITROGEN REMOVAL

Oxygen Profile

Theory indicates that for nitrogen removal a zone of the ditch must be maintained under anoxic conditions. In addition, a source of carbon should be added to this zone, which can be raw sewage or methanol. The optimum arrangements have not been demonstrated, but it is recommended that as a first trial approximately 50 percent of the ditch be maintained under anoxic conditions. Again, flexibility is needed in the aeration system for nitrogen removal applications.

MLSS and MLVSS

These considerations should be about the same as for conventional oxidation ditch plants and as outlined for nitrification. Some observation and experimentation will be required at each plant to develop operational guidelines for optimum nitrogen removal.

Carbon-Nitrogen Ratio

The biodegradable COD:N requirement for denitrification may be determined based on research conducted using methanol as an organic source. The methanol requirement essentially is that amount which will permit depletion of the oxygen in the dissolved nitrate and nitrite forms. The equation representing the methanol requirement is:

$$Cm = 2.47N_o + 1.53N_1 + 0.87D_o$$

Where Cm = required methanol concentration mg/l.

N_o = initial NO_3-N concentration mg/l.

N_1 = initial NO_2-N concentration mg/l.

D_o = initial DO, mg/l.

Methanol (CH_3OH) has a COD of 1.41 lb/lb methanol. Therefore, for a strictly nitrate nitrogen conversion a biodegradable COD to nitrate nitrogen ratio of 3.5 will be required for complete conversion.

pH

The denitrification process increases the alkalinity concentration, but normally will not totally offset the alkalinity lost by nitrification. The rest of the considerations are essentially the same as for nitrification.

Sludge Settleability

The comments for nitrification apply although some data suggest denitrified effluent does not settle as well as nitrified. In addition, if denitrification takes place in the final clarifier, gas will be released and sludge will tend to float and be carried over the effluent weir. Therefore, it is desireable to carry a positive oxygen level in the final clarifier and to remove sludge promptly from the final clarifier.

Sludge Recycle Ratio

The considerations are the same as for nitrification.

Sludge Wasting Rate

The considerations are the same as for nitrification.

Solids Retention Time

The considerations are the same as for nitrification except there is some indication that denitrification is rate limiting at cold temperatures. Under these conditions a higher SRT may be necessary for denitrification than for nitrification.

Temperature

The considerations are the same as for nitrification except for possible higher SRT during cold weather operation as discussed in the previous paragraph.

Hydraulics

If the plant hydraulic flow can be stabilized to some degree (for instance, with flow equalization) it will be easier to maintain the aerobic and anoxic zones necessary for nitrogen removal.

SUMMARY

Design of the plant for long aeration detention time and operation at relatively high MLSS of 4,000 to 6,000 mg/l should provide the long SRT necessary for year around nitrification and nitrogen removal. For winter operation the mixed liquor should be kept at the highest possible concentration and pH should be kept in the range of 7.5 to 9.0.

For nitrification only, adequate aeration should be provided to maintain a positive oxygen level in all parts of the ditch.

For nitrification plus nitrogen removal the aeration must be controlled so that at least one anoxic zone is maintained in the ditch. This will require a high degree of operator attention. It is questionable whether an automatic control system could satisfactorily control the formation of the required anoxic zone with any degree of reliability.

The multiple concentric channel type plant with aeration discs probably possesses some advantages for nitrogen removal applications, but this has not be demonstrated.

A properly designed oxidation ditch plant is capable of attaining essentially complete nitrification except when the mixed liquor temperature is near freezing.

Nitrogen removal of 70 to 80 percent can be achieved in oxidation ditch plants operating at temperatures above 15°C with proper design and operator control. At lower temperatures, 40 to 70 percent nitrogen removal is possible. This has been demonstrated at several oxidation ditch plants, which were properly designed and operated.

SECTION 5

OPERATION

OXIDATION DITCH PLANT PROBLEMS

As shown in Tables 5, 7, and 8, over 30 oxidation ditch plants were visited, contacted by letter or telephone, or reviewed in literature articles during this study. In each case particular attention was directed toward identifying various operational, maintenance, and process features and problems. It is felt that the comments in this section represent an excellent cross section of actual plant experience as related by responsible plant personnel.

Process and Process Operation

The most serious process operation difficulties resulted from equipment related problems.

> *Observations based on this study indicate that, as a group, oxidation ditch plants can be operated by average personnel to produce above average performance results.*

Many of the plants visited were manned only during a single shift and, in many cases, for only a portion of a single shift. A number of plants received little or no attention during weekends. Assuming no mechanical malfunctions, the plants perform well for long periods (days to weeks at a time) with little or no operator attention. Most plants practiced regular or periodic sludge wasting. In some cases sludge wasting was not practiced, but this caused eventual or regular carryover of excess solids in the effluent.

Almost without exception, operators and administrative personnel were well satisfied with the plants. In most cases the plants were meeting state discharge requirements.

A study by EPA Region VII(22) of winter performance of secondary wastewater treatment facilities concluded "...in general, the facilities were not meeting the secondary treatment effluent definition on the average except for the oxidation ditch subset".

All plants visited exhibited good mixed liquor characteristics (by visual observations). Some plants reported more than desired carryover of solids in the effluent and it is believed that the cause was either excessive oxygenation or poor management of sludge return or wasting.

Many plants could reduce the level of oxygenation in the aeration basin without detrimental effect and reduce energy cost. There were no signs of lack of oxygen. One plant was cycling the aerator one hour on and one hour off and reported excellent results for over a year. Several other plants were considering this scheme to lower operating costs and reduce oxygen levels.

Where screening or comminution is not regularly and properly carried out, rags and debris tend to cause problems in the plant especially in the return sludge system.

One deep type (10 feet) single channel plant reported poor mixing in the ditch.

A number of plants lacked sufficient drying bed capacity which may have led to erratic sludge wasting practices. This problem was also reported by EPA Region VIII personnel.

A number of plants lacked proper laboratory facilities and equipment. Some even lacked a building. These facilities are necessary for proper operation and maintenance of a wastewater treatment plant.

Equipment Problems

As with any plant containing mechanical equipment, oxidation ditch plant equipment was subject to problems, malfunctions, and failures. The level of preventative maintenance is important; however, this study did not attempt to relate these problems to the level of preventative maintenance.

As a general observation, oxidation ditch plants are capable of long periods of operation without mechanical problems and appear to operate with a very high mean time between equipment failures. There are some things that can be done to increase reliability of these plants as discussed later.

The following is a general summary of mechanical problems as related by plant personnel.

1. Some plant personnel reported that drives trip out on momentary electrical failures and do not restart upon restoration of power. This causes problems with unattended operation and consideration should be given to maintained-contact type electrical control for drives such as lift pumps, aerators, sludge return pumps, and other items in cases where plants operate unattended. Time delays can be provided so all drives do not start at the same time.

2. Some troubles were reported with return sludge pumping. In general, centrifugal, non-clog pumps seemed to give good service especially when a separate potable grade seal water source was used. Air lift pumps were not common as would be expected at plants with mechanical aeration. Where possible, it worked well to return the sludge by gravity to the raw lift station and pump it to the ditch with the raw sewage. This increased the size of the lift pumps, but eliminated one set of pumping.

3. Comminutors were a continuing maintenance problem. They required regular cleaning and care. Some operators had so many problems they stopped using the comminutor and allowed unscreened sewage to flow into the ditch. Almost without exception this caused problems in the return sludge system by clogging the sludge return rate adjustment valve (generally a telescoping valve) or pumps. This altered the sludge return rate and caused process problems if not detected early. Oscillating type comminutors appear to give best service, but still require regular maintenance.

4. Problems have been experienced with continuous dissolved oxygen controls because probes require almost continuous cleaning.

5. A hole is drilled in the scum box at some plants to keep the box and scum pipe flushed continuously and to provide some dilution water into the scum pit.

6. At one plant a non-reinforced concrete block intermediate wall in a multiple concentric channel plant collapsed and had to be replaced. Such walls should be reinforced.

7. One plant operator suggested that sludge return pumps should be variable speed, however, there are less expensive methods to adjust the return activated sludge flow rate.

8. Access walks should be designed to avoid receiving spray from aerators even under various wind conditions. This spray creates hazardous conditions because of algal growths and freezing. This oversight created continuing maintenance problems and should be considered in all designs.

9. Sludge reportedly settled in channels at some plants where flow control walls were not installed as recommended by manufacturers.

10. There was some reported corrosion in final clarifiers which resulted in materials failures (bolts in particular). These units should be drained and inspected annually. Any corrosion should be removed and the area recoated.

11. Weirs are a maintenance problem and must be cleaned frequently; in some cases as often as daily.

12. Aerators and aerator drives account for a major portion of the mechanical problems. Most plants experienced the following aerator related problems every two to five years per aerator unit.

   a. Loss of some "teeth" from brush type aerators due to corrosion of bolts or damage sustained while handling the aerators is a common occurance. This generally is not a serious problem and can be repaired during periodic shutdowns. Some manufacturers have redesigned their aerators to minimize this problem.

   b. Bearing problems were reported in gear drives, line shafts, and aerator shafts. Experience would seem to indicate a bearing problem every 2 to 5 years per aerator. These problems result from poor selection of bearings, constant splashing of water onto the bearing, improper initial alignment, differential settlement of bearing support structures and similar problems. Some manufacturers have taken steps to reduce bearing problems by using self aligning bearings, double seals, and by providing water shields, but bearing problems can still be expected. The magnitude of the bearing problem is not excessive and normal plant maintenance programs can handle this problem.

   c. Flexible couplings between line shafts caused problems on multiple concentric ring type plants.

   d. At one plant some of the disc aerators loosened from the shaft and had to be shimmed and reclamped.

   e. Gear reducer output shaft seals need replacing about once a year at some plants.

   f. There are a number of plants were gear reducer failure was experienced within a year of plant startup. This was probably due to improper initial alignment or differential settling of the aerator support structure.

   g. A couple of plants have experienced aerator torque tube failure or excessive deflection with very long aerators. At one plant the tube failed, collapsing in the middle. One consultant now requires solid shafts on all aerators rather than hollow torque tubes. It may be well to avoid the use of very long aerators. If a wide ditch is necessary, the width can be spanned using two shorter aerators driven by a common drive.

   h. Protective covers around bearings, couplings and drive units are not provided at most installations. Spray from the aerators keeps these components wet and possibly contributes to short life. The "slingers" at each end of the aerator are ineffective because wind blows this "tail" of water sideways onto adjacent components such as bearings and drive units. Corrosion and grit on shafts probably contributes to short seal life. For maximum service life and minimum maintenance these components should be shielded from water spray. Some manufacturers are taking steps to modify their standard designs to provide spray baffles and shields.

i. Some drive configurations require the aerator to be lifted out of position to remove the gear drive. This is a difficult operation requiring a crane. Access for a mobile crane should be provided to all aerators or other lifting provisions designed into the plant.

Cold Weather Operations

Cold weather problems related to oxidation ditch plants appear to be minimal. Those which have been identified are listed. These comments are based on visits and literature(9,12,13,14,22).

1. Final clarifiers should be covered where this is typical practice for other types of plants.

2. In moderately cold areas the spray from aerators will freeze on adjacent structures, bearings, gear reducers, and like equipment making maintenance difficult. Drive components should be covered to provide shielding from spray or these drives mounted in isolated compartments.

3. In moderately cold areas some problems are reported from ice build-up on clarifier scum collection boxes and eventual jamming of skimmer mechanisms.

4. Problems were reported with freezing of spray around aerators. The problem is solved by covering aerators in moderately cold areas, providing heated covers in very cold areas, and providing heated buildings over the aerators or over the whole ditch in extremely cold areas. When installed inside buildings, a shield should be provided over aerators to control spray.

5. In areas with periods of very cold weather all equipment requiring regular maintenance or service should be housed.

6. Poor mixed liquor settling was experienced during winter months at several plants in cold weather locations. This was probably due to filamentous growths resulting from the low winter loadings.

OPERATION AND MAINTENANCE REQUIREMENTS

Actual operation and maintenance information was obtained from a number of operating oxidation ditch plants. This information was analyzed in order to develop O & M requirements as a function of plant size and to determine if any significant variations are related to specific manufacturers equipment. A summary of the O & M requirements was obtained from the plants listed in Table 12. O&M requirements for normal plant operation, nitrification, and nitrogen removal are shown in Table 13. These requirements include sludge handling and drying on outdoor drying beds. Note that labor and electric energy costs are shown in two forms; the actual field costs and calculated costs based on assumed unit rates. This additional information is used for purposes of comparisons in Section 7.

The O & M requirements were developed by the following categories:

1. O & M labor
2. Energy (essentially electrical)
3. Chemicals (primarily chlorine)
4. Maintenance materials
5. Other (miscellaneous supplies, training, and other incidentals)

Labor

Labor requirements include operation, maintenance, sampling, and laboratory analysis. Actual O & M labor rates varied from $3.00 to $10.00 per hour including fringe benefits with the smaller facilities generally experiencing the lower costs. Typically, 15 to 20 hours per week were required to perform the sampling and laboratory analysis. Some plants contract for outside sampling and/or laboratory services.

Field investigations indicated a wide range of O & M labor requirements but the information in Table 13 was developed based on average practices. Information was available from very few plants over 1.0 mgd in size, and from none over 5 mgd, therefore, portions of the curve are extrapolated using graphical methods.

Energy

The electrical energy requirements include electric drives for aerators and pumps, heating and other miscellaneous plant requirements as reported by operating plants. Energy cost varied from $0.02 to $0.03 per kwh for the plants surveyed. Energy requirements were also calculated based on the aerator sizes shown in Table 2 plus pumping, heating, lighting, and other uses as a cross check on actual field data. These calculated usages correlated very well with the actual usages reported from the field.

Chemicals

Chemical costs, principally for chlorine, varied widely. Chlorine requirements and costs were calculated as a cross check on the field data.

TABLE 12. SOURCES OF OXIDATION DITCH PLANT O & M INFORMATION

Facility	Design capacity, mgd	Reference
Battle Creek, NB	0.21	EPA Region VII
Bolivar, MS	1.34	EPA Region VII
Nixa, MS	0.41	EPA Region VII
Seymour, MS	0.26	EPA Region VII
Birch Tree, MS	0.07	EPA Region VII
Eaton, CO	0.22	EPA Region VIII
Morrison, CO	0.07	M & I Report(20)
Berthoud, CO	1.10	M & I Report(21)
Dillon, SC		
Pee Dee	0.80	Plant Visit
Maple Swamp	0.36	Plant Visit
Clayton County, GA		
Jackson Plant	1.00	Plant Visit
N.E. Plant	1.00	Plant Visit
Clarksville, TX	0.70	Plant Visit
Huntsville, TX	0.80	Plant Visit
Paris, TX	4.62	Plant Visit
Daingerfield, TX	0.70	Plant Visit
Whitewright, TX	0.25	Plant Visit
Susanville, CA	0.80	Letter
Burney, CA	0.44	CA WRCB
Kershaw, SC	0.50	Plant Visit
Wymore, NB	0.40	Letter

TABLE 13. O & M REQUIREMENTS, OXIDATION DITCH PLANTS, 1976

Type operation and size, mgd	Annual requirements									
	O & M labor			Electrical energy						
	Days	Actual field, $1000	@ $9/Hr. $1000	Actual usage, 1000 Kwh	Actual field, $1000	@$0.03/Kwh, $1000	Chemicals, $1000	Mainten-ance materials, $1000	Other, $1000	Total[*] $1000
NORMAL										
0.05	220	5.2	15.8	46	0.9	1.4	0.3	0.5	0.5	18.5
0.10	250	6.3	18.0	72	1.4	2.2	0.6	0.7	0.6	22.1
0.50	380	16.0	27.4	280	5.4	8.4	2.3	2.2	0.8	41.1
1.00	520	27.5	37.4	500	11.0	15.0	4.2	4.7	1.1	62.4
5.00	1,900	125.0	136.8	2,000	50.2	60.0	17.5	25.0	1.7	241.0
10.00 (**)	3,800	230.0	273.6	3,700	110.0	111.0	30.0	50.0	2.0	466.6
NITRIFICATION										
0.05	226	5.3	16.3	46	0.9	1.4	0.3	0.5	0.5	19.0
0.10	257	6.5	18.5	72	1.4	2.2	0.6	0.7	0.6	22.6
0.50	388	16.3	27.9	280	5.4	8.4	2.3	2.2	0.8	41.6
1.00	529	28.0	38.1	500	11.0	15.0	4.2	4.7	1.1	63.1
5.00	1,911	125.7	137.6	2,000	50.2	60.0	17.5	25.0	1.7	241.8
10.00 (**)	3,812	230.7	274.5	3,700	110.0	111.0	30.0	50.0	2.0	467.5
NITROGEN REMOVAL										
0.05	310	7.3	22.3	37	0.7	1.0	0.3	0.5	0.5	24.7
0.10	340	8.6	24.5	58	1.1	1.7	0.6	0.7	0.6	28.1
0.50	490	20.6	35.3	224	4.3	6.7	2.3	2.2	0.8	47.3
1.00	630	33.3	45.4	400	8.8	12.0	4.2	4.7	1.1	67.4
5.00	2,030	133.6	146.2	1,600	40.2	48.0	17.5	25.0	1.7	238.4
10.00 (**)	3,930	237.9	282.9	2,960	88.0	88.8	30.0	50.0	2.0	453.7

(*) Includes Labor at $9.00/Hr and Electric Energy at $0.03/Kwh.

(**) Data extrapolated between 5 and 10 mgd graphically.

The calculated costs were based on a feed rate of 10 mg/l and a chlorine cost ranging from $0.11 to $0.22 per pound varying with plant size from large to small respectively.

Maintenance Materials

The cost for maintenance materials varied widely from plant to plant and little information was available for the large plants. The information in Table 13 is quite reliable in the range of plant size from 0.1 to 1.0 mgd and is extrapolated outside these limits using graphical methods.

Other

The "other" category includes miscellaneous costs related to O & M including minor supplies, training, and similar items.

NITRIFICATION

Only slight incremental increased O&M costs are required for nitrification optimization over those for a normal extended aeration oxidation ditch plant. Some low alkalinity wastewaters may require continuous pH adjustment, but these are considered special cases.

The total incremental addition to plant O & M costs would be 6 to 12 additional days of operator time per year for monitoring pH. This is shown in Table 13.

NITROGEN REMOVAL

The primary incremental O & M considerations for nitrogen removal would be additional operator attention to maintain the anoxic zone and some possible electrical energy savings due to more controlled aeration and prevention of over aeration. There is no actual plant information to serve as a basis for the determination so the incremental O & M modifications are based on estimates and calculations. It is estimated that aerator energy use could be decreased by 20 percent with the careful control of aeration required by single stage nitrogen removal operation. The savings might be much higher in some special cases. Additional operator attention is required to reliably maintain the anoxic zone and this is reflected by higher O & M labor requirements. The additional time will vary somewhat with plant size because the larger plants generally have more aerators and are spread out more. It is estimated that the annual additional operator time will vary from 90 man days for small plants to 130 man days for the larger plants including any nitrification considerations. The incremental O & M requirements are shown in Table 13. The incremental labor requirements shown may seem excessive in relation to basic requirements, but operating the plant for consistent nitrogen removal in the 70 to 80 percent range will require significant operator attention both during the week days and on weekends. Operation and maintenance requirements assume that no methanol feed is required for nitrogen removal.

SECTION 6

CONSTRUCTION

CONSTRUCTION COST

Oxidation ditch plant construction costs were determined from costs of recent plants constructed as shown in Table 14 and include a building with appropriate laboratory.

The construction costs are shown in Figure 25 with an estimated breakdown of construction costs shown in Table 15. The breakdown of costs was determined from bid tabulations obtained by letter and during plant visits. All costs are referenced to the EPA treatment plant index of 262.3 for third quarter of 1976. Construction costs do not include land, engineering, legal, or financing during construction.

NITRIFICATION CONSTRUCTION COSTS

The construction costs for an oxidation ditch plant designed for nitrification should be essentially the same as the standard oxidation ditch plant.

NITROGEN REMOVAL CONSTRUCTION COSTS

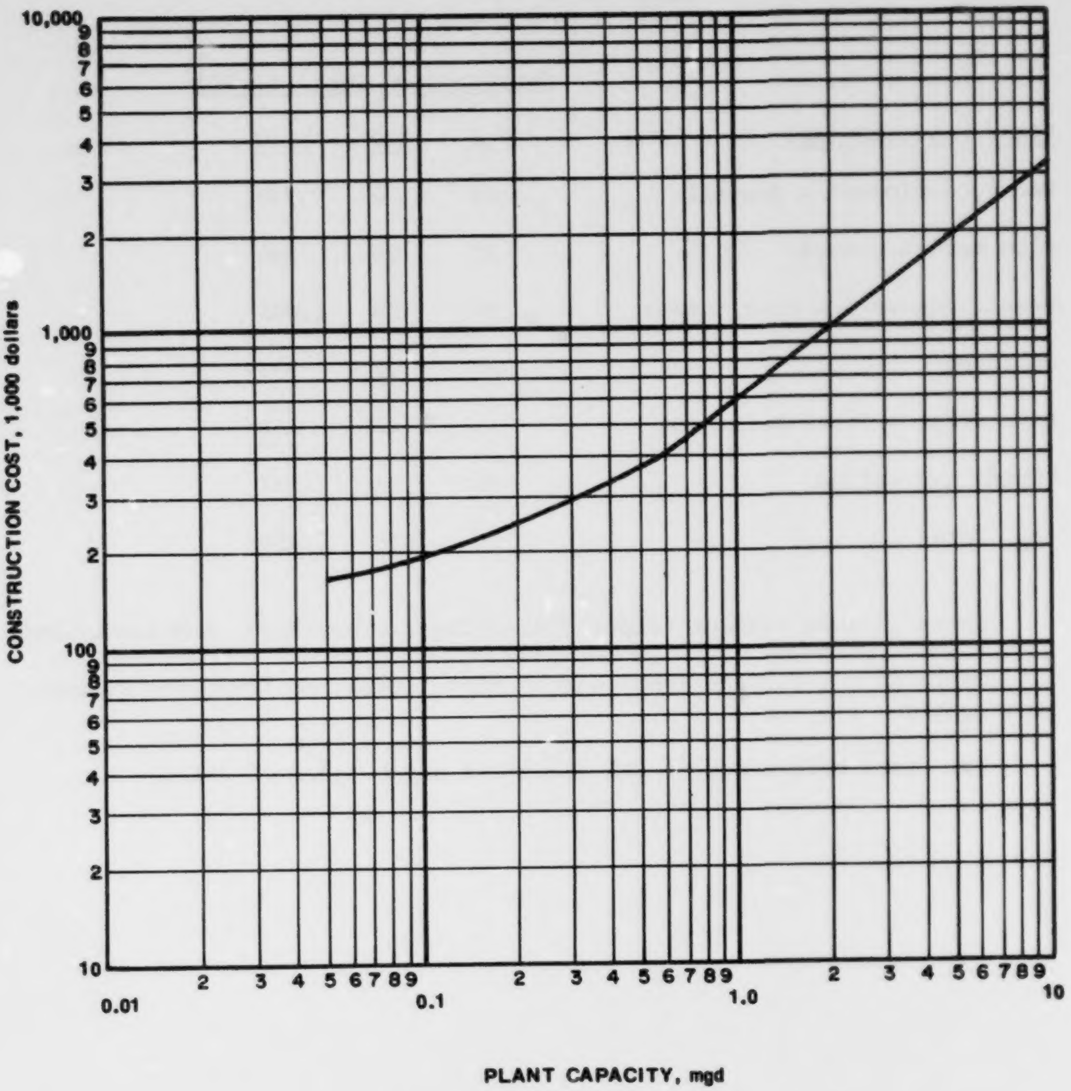
Any additional construction costs related to nitrogen removal in a single oxidation ditch channel would be only for permanent ditch dissolved oxygen monitoring and control equipment, provisions for easy adjustment of the ditch oxygen level, any necessary auxiliary flow velocity devices, and, perhaps, a more conservative final clarifier rating because of the possibility of some nitrogen gas release in the clarifier.

It is doubtful whether an automatic dissolved oxygen control loop will give satisfactory performance in this application and, therefore, questionable whether a permanent installation should be made. The best arrangement may be the use of a portable instrument, manual measurements, and manual adjustment of the aeration.

The aeration equipment should be readily adjustable in order to facilitate changing the oxygen transfer rate. The effluent weir can be designed so the ditch level will change with flow rate and thereby change the oxygen transfer rate to partially compensate for flow variations. It is suggested that the aerator be equipped with a variable speed drive to provide easy manual adjustment of the aerator oxygen transfer rate. It is also possible

TABLE 14. SOURCES OF OXIDATION DITCH PLANT CONSTRUCTION COST INFORMATION

Facility	Design capacity, mgd	Year constructed	Original cost, $1000	Reference
Bossier City, LA	8.00	1976	3,275	Lakeside
Tivoli, TX	0.08	1976	179	Lakeside
Bernie, MO	0.44	1976	326	Lakeside
Glen Rose, TX	0.35	1976	248	Lakeside
Beeville, TX	2.50	1976	1,568	Lakeside
Del Rio, TX	1.00	1975	534	Lakeside
Burkburnett, TX	1.10	1975	369	Lakeside
Henrietta, TX	0.28	1975	207	Lakeside
Woodsboro, TX	0.20	1975	118	Lakeside
Shiner, TX	0.60	1975	300	Lakeside
Glenpool, OK	0.25	1974	220	Lakeside
Crofton, NB	0.25	1974	209	Lakeside
Diboll, TX	1.00	1973	225	Lakeside
Tolar, TX	0.10	1972	128	Lakeside
Mexia, TX	0.45	1972	150	Lakeside
Del Rio, TX	0.40	1972	150	Lakeside
Wylie, TX	1.00	1972	223	Lakeside
Seymour, MO	0.25	1971	94	Lakeside
Marion, S.C.	1.00	1969	412	Plant Visit
Greensboro, GA	0.35	1971	322	Plant Visit
Cleveland, GA	0.33	1969	80	Plant Visit
Baldwin, GA	0.30	1974	165	Plant Visit
Clayton Co., GA				
Jackson Plant	1.00	1974	1,102	Plant Visit
NE Plant	1.00	1971	431	Plant Visit
Clarksville, TX	0.70	1975	311	Plant Visit
Huntsville, TX	0.80	1971	916	Plant Visit
Paris, TX	4.62	1972	1,236	Plant Visit
Daingerfield, TX	0.90	1971	198	Plant Visit
Whitewright, TX	0.25	1971	130	Plant Visit
Big Lake, CA	0.15	1973	246	State
June Lake, CA	1.00	1973	408	State
Bar Harbor, ME	0.07	1973	139	Letter
Indian Island, ME	0.07	1974	382	Letter
Newtown Co., GA	0.50	1974	344	Plant Visit
Conyers, GA	1.00	1975	569	Plant Visit
Kershaw, S.C.	0.50	1974	385	Plant Visit
Nixa, MO	0.37	1970	95	Lakeside
Salem, MO	0.74	1970	187	Lakeside
Rushsylvania, OH	0.10	1970	57	Lakeside
Wheelersburg, OH	1.20	1970	560	Lakeside
Lucasville, OH	2.20	1970	607	Lakeside
Greenville, OH	3.30	1970	610	Lakeside
San Antonio, TX	0.16	1970	40	Lakeside
Vernon, TX	1.00	1970	185	Lakeside



Cost includes building, laboratory outdoor sludge drying beds, but excludes land, engineering, legal and financing during construction. All costs referenced to EPA treatment plant index 262.3, fall 1976.

Figure 25. Oxidation ditch plant average construction cost

TABLE 15. OXIDATION DITCH PLANT CONSTRUCTION COST, 1976

	Construction Cost, $1,000		
Plant Capacity, mgd	0.1	1.0	10.0
Manuf. equipment & install.	49	138	770
Site work & general	27	96	536
Reinf. concrete & misc. steel	55	180	1,005
Building	16	42	234
Electrical & instrumentation	19	48	268
Piping and valves	29	96	537
Total	195	600	3,350

Costs include outdoor sludge drying beds, a building, and laboratory.

Costs do not include land, engineering, legal, or financing during construction.

All costs referenced to EPA treatment plant index of 262.3

to apply on-off operation of the aerator, but this technique has not been widely demonstrated in the field and, therefore, would be experimental.

The need for auxiliary flow velocity devices would depend on the application. The use of these devices has been demonstrated at Glenwood, Minnesota using a custom built floating device. It is possible that commercially available propeller mixers could be adapted to this service.

It is recommended that the final clarifier be rated for an overflow rate of 300 to 400 gpd/sq ft in nitrogen removal applications. The clarifiers should also have deep scum baffles and double skimmer bars.

The estimated cost effect of these recommendations is shown in Table 16.

EQUIPMENT PRICING

Pricing was obtained from the manufacturers for aerators, clarifier mechanisms, and effluent weirs and is shown in Table 17. These costs include freight, but not installation. Costs are shown for two cases; with minimum equipment and with at least dual aerators and clarifiers. Where dual units are used each is sized for one half of design flow.

As a general rule, the small and large diameter aerators are very similarly priced per foot of length with the small diameter being less than 10 percent below the price of the large diameter aerator.

DITCH CONFIGURATION

Construction cost estimates were made for various single channel oxidation ditch configurations for comparative purposes. The configurations include shallow sloped and straight sides and deep straight side ditches. These estimates were all based on the same assumptions using the Richardson Estimating Guide(27) and, therefore, should be readily comparable. The results are shown in Table 18. Costs do not include land, engineering, legal, or financing during construction, but include 25 percent contractor profit and overhead.

The effect of configuration is not highly significant but indicates the shallow sloped side ditch is cost effective for small plants and either the shallow sloped side ditch or deep ditch is suitable for intermediate plants. The deep configuration would probably be used for large plants.

PLANT AREA

The space requirements for various oxidation ditch plants will vary widely, however the typical designs shown in Table 2 were used to develop the average area requirements as shown in Table 19. These do not represent the absolute minimum size, but the area needed for a reasonable and workable layout. The area requirements include space for the raw sewage lift station, headworks, oxidation ditch, final clarifier(s), return sludge

TABLE 16. INCREMENTAL CONSTRUCTION COSTS FOR NITROGEN REMOVAL

	Construction cost, $1000			
Plant capacity, mgd	0.1	0.5	1.0	10.0
Aerator variable speed drive	2.0	10.0	14.0	112.0
Auxiliary flow devices	2.0	4.0	6.0	32.0
Final clarifier modifications	3.0	5.0	9.0	40.0
Total incremental addition	7.0	19.0	29.0	184.0
% of standard plant construction costs	3.6	5.0	4.8	5.5

These costs would be additive to the construction costs for a standard oxidation ditch plant shown in Table 15 and are based on the same assumptions.

TABLE 17. OXIDATION DITCH PLANT MAJOR EQUIPMENT COST, 1976

	Plant capacity, mgd		
	0.1	1.0	10.0
Clarifier mechanism(s)			
Number (minimum number units)(*)	1	1	2
Size, diameter, ft.	16	50	112
Cost, $	13,000	30,000	230,000
Number (dual)(**)	2	2	2
Size, diameter, ft.	12	36	112
Cost, $	23,000	40,000	230,000
Aerator(s)			
Number (minimum number units)(*)	1	2	10
Length, ft.	6	24	50
Cost, $	9,000	36,000	320,000
Number (with at least dual units)(**)	2	2	10
Length, ft.	4	24	50
Cost, $	16,000	36,000	320,000
Adjustable effluent weir			
Cost, $	1,700	3,000	-
Total, minimum equipment	$23,700	$69,000	$550,000
Total, dual equipment	$40,700	$79,000	$550,000

(*) Based on minimum of one unit
(**) Based on minimum of two units
Prices include an allowance for freight, but no installation

TABLE 18. COMPARATIVE CONSTRUCTION COST OF OXIDATION DITCH CONFIGURATIONS

Design capacity, mgd	Cost,$	Design assumptions
Shallow sloped side (6 feet deep)		
0.1	30,621	4" thick poured concrete with welded wire reinforcing. Concrete poured against undisturbed earth. Sloped sides poured without use of face forms.
0.5	85,368	
1.0	133,589	
Shallow straight side (6 feet deep)		
0.1	35,516	4" thick concrete bottom with welded wire reinforcing poured against undisturbed earth. Vertical walls 8" concrete with rebar and concrete footings. Center wall common to both sides of ditch.
0.5	98,249	
1.0	144,770	
Deep straight side (12 feet deep)		
0.1	33,699	6" thick reinforced concrete bottom poured against undisturbed earth. Vertical walls 12" concrete with rebar and concrete footings. Center wall common to both sides of ditch.
0.5	76,605	
1.0	132,464	

Estimates based on Richardson Estimating Guide(27) and contain 25% contractor profit and overhead, but exclude land, engineering, legal, and financing during construction. Assumed dimensions are shown in Table 2.

TABLE 19. APPROXIMATE OXIDATION DITCH PLANT AREA REQUIREMENTS

Plant capacity, mgd	Area, acres
0.05	0.07
0.1	1.2
0.5	2.8
1.0	4.2
5.0	10.0
10.0	17.0

Area includes space for raw sewage lift station, headworks, oxidation ditch, final clarifier(s), return sludge pumping, sludge drying beds, access roads, control building, and a 40 foot buffer around plant perimeter. Space is not provided for future expansion.

pumping, sludge drying beds, access roads, control building, and at least a 40 foot buffer around the plant perimeter. Area requirements do not provide space for plant expansion.

COLD CLIMATE

In extremely cold climates it may be necessary or desireable to enclose the final clarifier. In addition, all equipment requiring periodic maintenance or service should be enclosed including aerators. In very severe climate it may be desirable to cover the entire oxidation ditch, but experience in Fairbanks, Alaska(12,13,14) indicates that ditches can operate in extremely cold climates without an overall cover.

OTHER FACTORS

There are few other factors which have an impact on oxidation ditch plant construction costs because the plants are generally conservatively designed. Influent nitrogen concentration above 60 mg/l would require additional aeration capacity, but nitrogen concentrations will seldom be that high.

SECTION 7

COMPETING PROCESSES

GENERAL

This section contains process performance and cost information on treatment processes which are competitive to oxidation ditch plants. These competitive processes include biological treatment and physical-chemical nitrogen removal. The biological treatment also includes biological nitrification and denitrification.

The performance and cost information was developed based on actual plant data to the extent possible. Portions of the cost data are based on other published cost information as referenced in the text.

CAPABILITY AND RELIABILITY OF COMPETING BIOLOGICAL PROCESSES

Treatment capabilities of the following processes are covered in this section:

1. Trickling Filters

2. Activated Sludge - with and without effluent filtration

3. Rotating Biological Discs - with and without chemical coagulation

Performance data is presented in two forms. The first form is expected average process performance which is simply an averaging of plant performance over a period of time.

The second form is the return frequency or probability of occurance of specific effluent qualities. This form presents a more complete and realistic assessment of the capabilities of a specific plant or process because it includes the high and low peaks. Oxidation ditch plant data as previously developed are plotted on the same figures for comparative purposes.

Trickling Filters

Average data reported in the Deeds and Data section of the Journal of Water Pollution Control Federation (JWPCF) between 1960 and 1965 are summarized on Figure 26. Organic loading for these plants ranged from 5.7 to 100 pounds of BOD_5/1,000 cu ft/day.

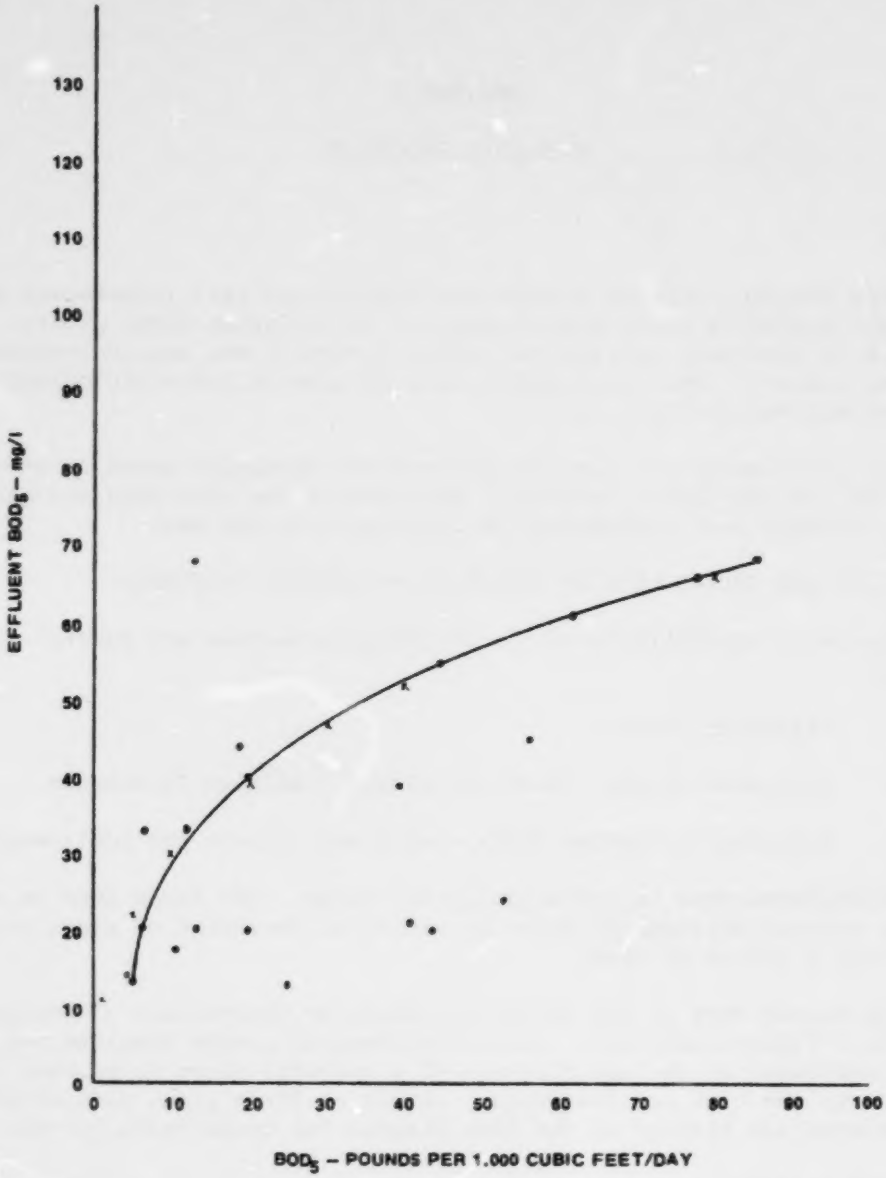


Figure 26. Effluent quality, trickling filters.

Typical variations in effluent quality from two Texas trickling filter plants, one low rate and one high rate, are shown in Figure 27. The data for trickling filter plants in Figure 27 were selected from several reports and are a fairly typical representation of the capability of this process in a relatively warm climate. In specific instances, an effluent quality of 25 mg/l and nitrification to levels of 2 mg/l NH_3-N have been achieved.

Benzie, et al(28), showed decreasing BOD_5 removal efficiencies due to cold weather operation of 17 trickling filter plants in Michigan. The loss in efficiency varied considerably depending on recirculation practices. Averaging the data of all 17 plants, the loss in BOD_5 removal efficiency was 12%. The decrease in efficiency for those plants employing recirculation was 21%. Those plants without recirculation showed a 6% decrease in efficiency. The analyses show the 21% difference to be statistically significant but the 6% difference was not.

Culp/Wesner/Culp visited 13 trickling filter plants of various designs as part of the work under EPA Contract 68-01-4329. One to three years data were obtained from each of these plants and are shown in Table 20.

Trickling filter plant data summarized from EPA Region inspection and technical assistance reports are shown in Table 21.

Activated Sludge

The activated sludge process is capable of converting essentially all influent soluble organic matter to solids. It is necessary to efficiently remove these solids in order to attain high quality effluents. Therefore, the final clarification unit process is extremely important to production of a high quality effluent. Careful operation is necessary to attain good effluent quality consistently.

The data from activated sludge processes reflect the problems in attaining consistently good effluent quality. The deeds and data section of the JWPCF reports data from 20 plants during the period from 1960 to 1965. Plant BOD loadings ranging from 18 to 74 pounds BOD_5/1,000 cu ft/day resulted in average effluent BOD_5 values of 3 to 86 mg/l with 8 of the 20 plants reporting average BOD_5 values of less than 20 mg/l.

Activated sludge plant data summarized from EPA Region inspection and technical assistance reports are shown in Table 22. Data are shown for conventional, contact stabilization, and extended aeration process variations.

Daily data presented in the form of return frequency are shown on Figure 28 and were developed from 4 Austin, Texas contact stabilization plants and activated sludge plants at Grand Island, Nebraska; Dallas, Texas; High Point, North Carolina; Chicago, Illinois; and Ypsilanti, Michigan. The data, in each case, represent an entire year. The plants selected for data presentation were those where good analyses are known to be performed, a variety of activated sludge processes are used and a range

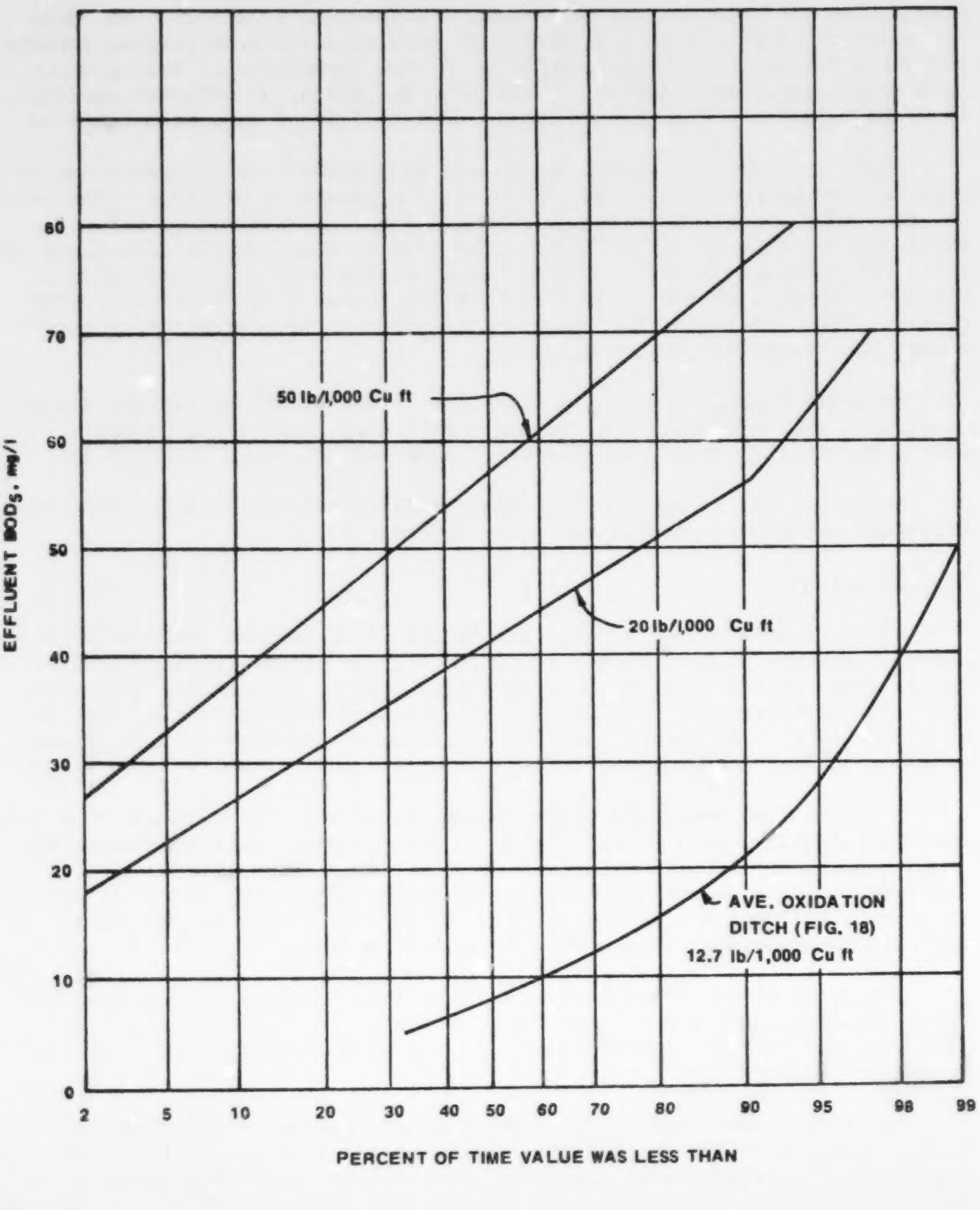


Figure 27. Trickling filter effluent quality, two Texas plants.

TABLE 20. TRICKLING FILTER SUMMARY, PLANT VISITS

Location	Design flow, mgd	Average flow, mgd	Average effluent			
			BOD$_5$ mg/l	BOD$_5$ Removal, %	TSS, mg/l	TSS removal, %
Iowa:						
Shellsburg	0.0825	0.0609	47	70	-	-
Center Pt.	0.200	0.141	43	72	-	-
Monticello	0.800	0.412	39	80	-	-
Cascade	0.220	0.081	48	76	-	-
Independence	0.750	0.889	85	87	-	-
Lakeview	0.175	0.153	69	69	-	-
Georgia:						
Westside, 1975	1.00	1.059	25	75	30	70
Westside, 1976	1.00	0.971	35	72	38	68
Kennesaw, 1974	0.30	0.27	24	85	28	87
Sandtown, 1976	1.00	1.222	51	70	50	60
Newman, 1975	0.40	0.328	30	88	28	88
Newman, 1976	0.400	0.346	23	88	29	86
Intr. Cr., 1975	20.0	13.9	40	82	26	78
Intr. Cr., 1976	20.0	13.1	35	83	31	74
College Pk., 1976	1.2	1.36	43	90	35	76
Athens #1, 1975	5.00	5.60	80	69	64	73
Athens #1, 1976	5.00	5.14	64	73	47	80
Athens #2, 1975	2.00	2.90	46	73	58	68
Athens #2, 1976	2.00	2.60	47	75	40	78
Cedartown, 1974	1.00	0.82	46	77	40	80
Cedartown, 1975	1.00	0.92	17	89	18	90
Cedartown, 1976	1.00	1.06	23	88	22	88
	Average removal			79		78

Data obtained during plant visits under EPA Contract 68-01-4329 by Culp/Wesner/Culp.

TABLE 21. TRICKLING FILTER SUMMARY, EPA REGION DATA.

Location	Organic loading, lb BOD_5/day/ 1,000 cu ft	Flow, mgd		Influent, mg/l		Effluent, mg/l		Removal, %	
		Average	Design	BOD_5	TSS	BOD_5	TSS	BOD_5	TSS
Frederick, MD	High rate	4.6	7.0	351	161	100	40	72	76
Chapel Hill, NC	35	2.8	3.0	179	247	36	36	80	85
Marlboro, MA	74	2.22	2.0	134	112	62	32	54	71
Richardson, TX	13	1.5	3.0	166	155	20	15	88	90
Allentown, PA	18	24.0	24.0	190	215	40	40	79	81
Pueblo, CO	42	14.6	14.0	148	163	39	29	74	82
Madera, CA	Del Pak	2.2	7.0	137	121	30	27	78	78
Mt. Vernon, CA	High rate	2.4	3.0	116	155	22	35	81	78
Santa Maria, CA	Two stage	4.4	6.5	200	280	33	45	84	84
Chinook, MT	High rate	0.22	0.5	168	179	71	59	58	67
Bountiful, UT	Std. rate	5.3	5.4	170	175	22	22	87	88
Englewood, CO	-	8.5	12.0	258	207	32	29	88	86
Murray, UT	-	4.9	8.0	193	193	36	35	81	82
Ft. Morgan, CO	Two stage	1.5	3.6	917	645	43	44	95	93
					Average removal			79	82

Data obtained from EPA Region inspection and technical assistance reports.

TABLE 22. ACTIVATED SLUDGE SUMMARY, EPA REGION DATA

Location	Flow, mgd		Influent, mg/l		Effluent, mg/l		Removal, %	
	Ave	Design	BOD_5	TSS	BOD_5	TSS	BOD_5	TSS
CONVENTIONAL								
Aspen, CO	1.1	2.0	336	298	19	20	94	93
Missoula, MT	5.5	10.0	133	129	54	48	59	63
Snowmass, CO	0.8	0.8	185	180	34	53	82	71
Havre, MT	1.3	1.8	275	275	31	30	89	87
Barstow, CA	1.7	4.5	150	250	16	25	89	90
W. Sacramento, CA	3.5	2.5	140	130	20	27	86	79
Carmel, CA	2.5	3.0	200	300	20	30	90	90
Monterey, CA	3.5	4.6	250	250	35	70	86	72
Zephur Cove, NV	1.0	3.0	334	237	12	11	96	95
Fairfield, OH	1.3	1.0	173	172	26	26	85	85
Grand Rapids, ND	38	-	100	140	18	40	82	71
					Average removal		85	81
CONTACT STABILIZATION								
Richwood, WV	0.39	0.5	89	94	30	33	67	65
Pen Argyl, PA	1.1	0.95	162	75	16	26	90	62
Lolo, MT	0.15	0.25	233	172	17	12	93	93
Baden, PA	0.55	0.5	206	202	22	21	89	90
Lancaster, PA	9.24	12.0	168	227	32	35	81	85
Manteca, CA	1.3	3.2	200	180	50	25	75	86
Savage, MD	5.9	-	158	206	22	41	86	80
Coralville, IA	0.9	-	135	150	26	24	81	84
					Average removal		83	81
EXTENDED AERATION								
Athens, WV	0.15	0.25	129	258	14	33	88	84
Follansbee, WV	0.5	0.5	200	190	16	48	92	75
Cheyanne, WY	0.43	0.8	262	272	-	-	-	-
Elizabeth, CO	-	0.065	441	341	30	96	93	72
Elbert, CO	0.007	0.025	208	125	16	27	92	78
					Average removal		91	77

Data obtained from EPA Region inspection and technical assistance reports.

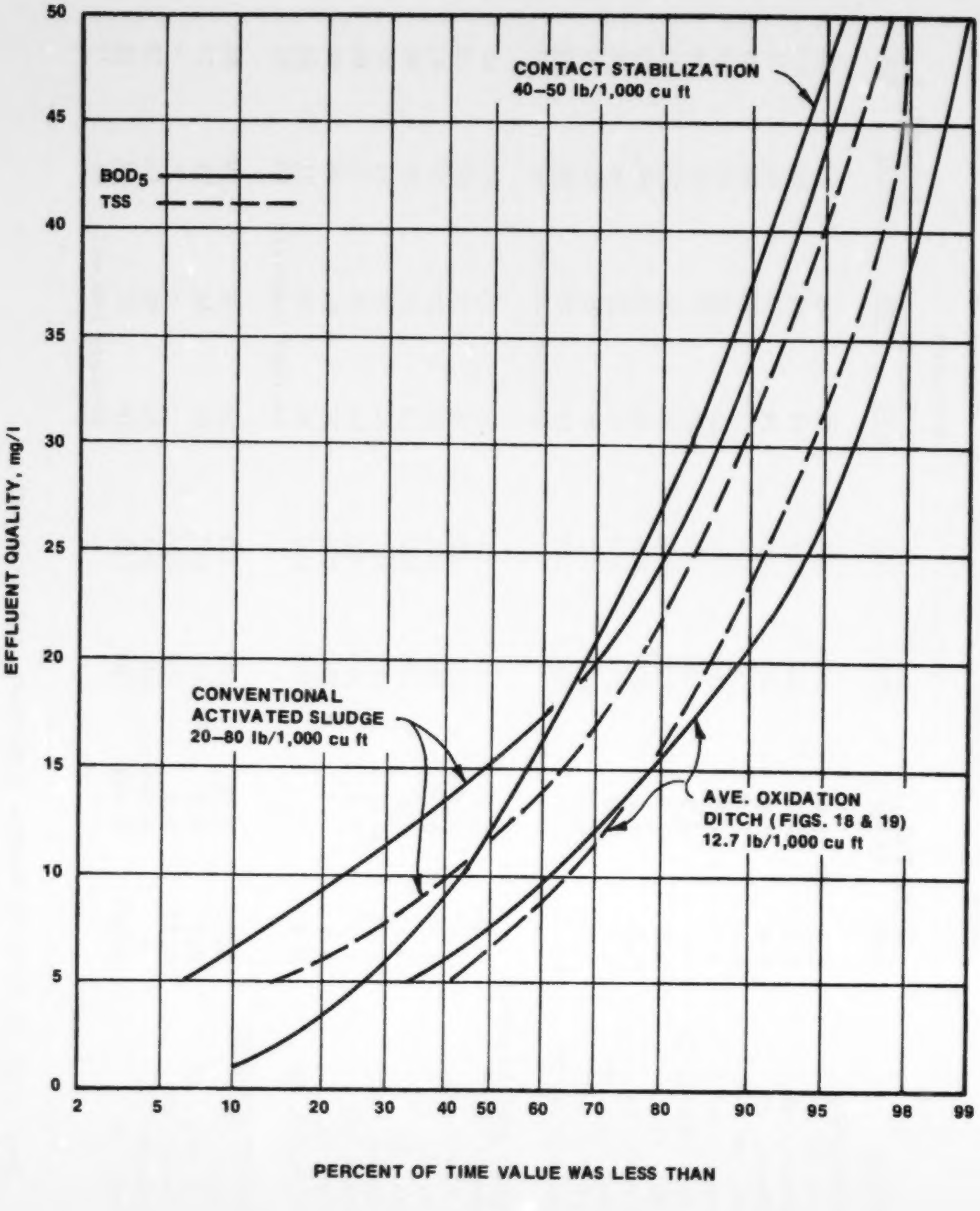


Figure 28. Activated sludge effluent quality.

of loadings are experienced. The conclusions which may be made from these data are:

1. Two of the activated sludge plants treat significant industrial waste flows. The High Point, North Carolina, Eastside plant receives textile dye wastes and the Grand Island, Nebraska plant receives slaughterhouse wastes. Both plants perform as well as the other domestic waste plants. Therefore, activated sludge plants can be designed and operated to treat industrial wastes and perform as well as plants treating little or no industrial wastes.

2. The plant loadings range from 20 to 80 pounds of BOD_5/day/1,000 cu ft of aeration tank volume. The performance of the plants is not related to unit aeration basin organic loading.

3. Whereas all of the plants from which data are used are considered to have good operational control and design, the Grand Island plant, for one year, produced an effluent BOD_5 significantly better than 10 mg/l, 70 percent of the time. Four of the plants produced BOD_5 effluent better than 35 mg/l, 90 percent of the time.

The potential for the activated sludge process is exemplified by the Grand Island plant which produced a BOD_5 better than 5 mg/l, 50 percent of the time and 20 mg/l, 90 percent of the time.

Biological nitrification of ammonia to nitrate is a well established phenomenon and several bench scale processes and demonstration processes have shown virtually complete conversion is possible if sufficient oxygen transfer is available. Several activated sludge plants having excess oxygen transfer capability do nitrify; however, until the past few years, few plants routinely monitored effluent ammonia.

A source of good data suitable for probability analysis on activated sludge nitrification is available from the Dallas demonstration pilot plant as shown in Figure 29. The plant was a constant flow plant receiving trickling filter effluent having an average BOD_5 of 60 mg/l. The aeration basin was loaded at 20 pounds/1,000 cu ft/day and had an average hydraulic detention time of 4 hours. Solids retention time (SRT) varied from 7 to 20 days. The final clarifier had an overflow rate of 350 gpd/sq ft and a 3.5 hour detention time.

The median effluent BOD_5 was less than 20 mg/l and 50 percent of the time the effluent contained zero ammonia nitrogen. Seventy percent of the time the effluent ammonia was less than 2 mg/l. Poorer results were obtained when the SRT was in excess of 15 days. Clarifier solids buildup associated with attempting to thicken sludge in the clarifier resulted in nitrification and poorer effluent quality. The pilot plant was monitored continually and the operators were highly skilled individuals who reacted quickly to ill effects.

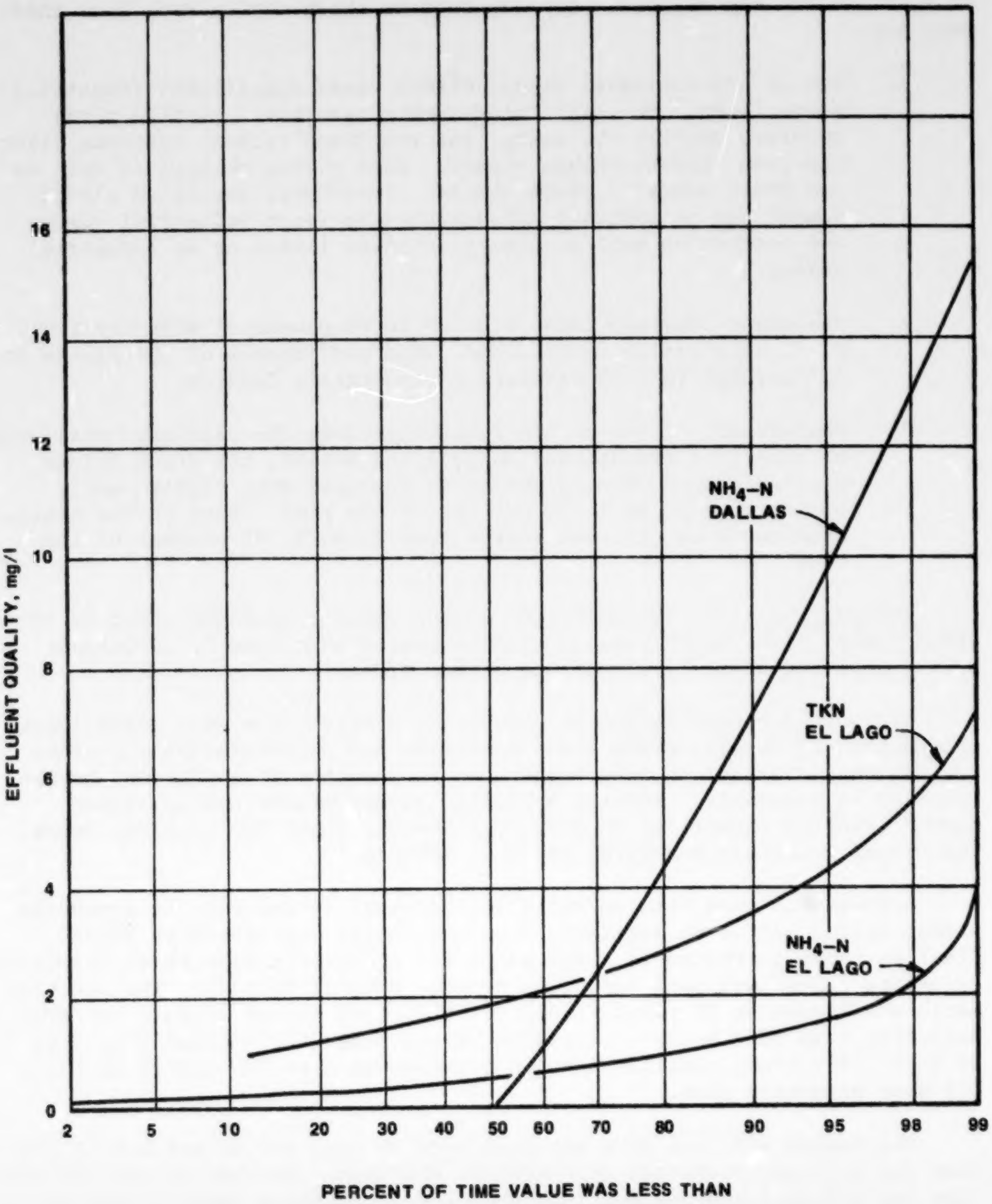


Figure 29. Activated sludge effluent quality, Dallas, Texas nitrification pilot plant and El Lago, Texas.

The data show that the activated sludge process may produce an effluent quality of 2 mg/l NH_4-N 70 percent of the time.

The low rate trickling filter plant at El Lago, Texas was modified in 1973 to provide advanced waste treatment. A second stage suspended growth reactor was added down stream of the trickling filter to provide essentially two stage nitrification. Small media biological denitrification towers were added to provide denitrification using methanol as a carbon source. Performance results for this plant are shown in Figure 29. Although the El Lago treatment process is not totally activated sludge, the data do provide a good indication of biological nitrification-denitrification performance.

Polishing of activated sludge effluent using filtration can produce a high effluent quality. Soluble BOD_5 from the activated sludge process is low and the majority of the remaining BOD_5 results from the solids escaping the final clarifier.

When no coagulants are used, the filterability of solids in a biological plant effluent is dependent upon the degree of flocculation achieved in the biological process. A trickling filter achieves a poor degree of flocculation and efficient filtration of the effluent from a trickling filter plant will usually provide only about 50 percent removal or less of the suspended solids normally present. The activated sludge process is capable of much higher degrees of biological flocculation than is the trickling filter process. Culp and Hansen(8) found that up to 98 percent of the suspended solids in an extended aeration plant effluent with 24 hr aeration treating domestic sewage could be removed by filtration producing turbidities as low as 0.3 Formazin Turbidity Unit (FTU) without the use of coagulants. These authors later reported that pilot plant studies showed that the degree of biological flocculation achieved in an activated sludge plant was directly proportional to the aeration time and inversely proportional to the ratio of the amount of organic material added per day to the amount of suspended solids in the aeration chamber (F/M ratio). Variation of mixed liquor suspended solids in the normal operating range of 1,500-5,000 mg/l did not significantly affect the filterabiltiy of the effluent at a given aeration time and load factor. For domestic wastes, aeration times of 10 hr or more were found to provide flocculation adequate to permit an efficient downstream filter to remove 90-98 percent of the effluent suspended solids. The flocculation provided by aeration times of 6-8 hr with domestic wastes enabled 70-85 percent suspended solids removal from the secondary effluent.

Data from four activated sludge plants with effluent filtration are shown in Figure 30. The data represent periods from 20 days to one year of operation. The average and best oxidation ditch unfiltered effluent BOD_5 are shown for comparison.

The data are fairly consistent, indicating an effluent BOD_5 of less than 5 mg/l, 50 percent of the time, and less than 10 mg/l 90 to 95 percent of the time.

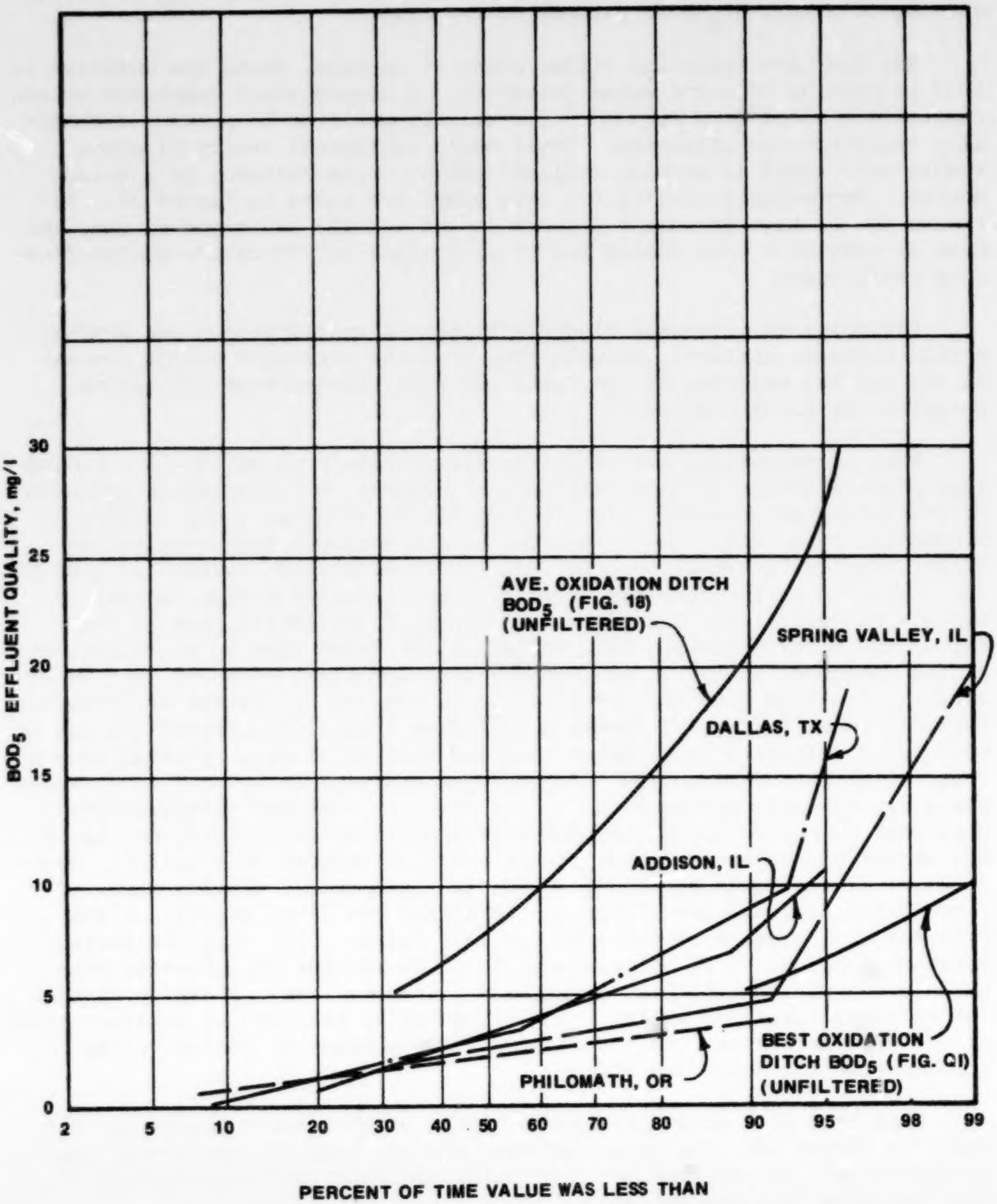


Figure 30. Filtered activated sludge plant BOD$_5$ quality based on four plants.

The current Dallas effluent criteria require an effluent quality of 10/10 (BOD_5/TSS). Since the effluent filtration of activated sludge provided better quality than the criteria, effluent filters were installed at Dallas to treat only a portion (70%) of the flow and the effluent was blended to meet effluent quality requirements. This resulted in cost savings over providing filtration of the entire flow.

A survey of small privately operated extended aeration and contact stabilization package plants was carried out in the Cincinnati, Ohio, area and in Dade County, Florida. These plants were treating domestic sewage and were in the size range of 0.05 to 1.0 mgd.

The median final effluent quality was reported as follows:

	Dade Co.	Cincinnati
No. of Plants	46	20
BOD_5, mg/l	13	29
Suspended Solids, mg/l	26	42

The reported reliability is shown in Figures 31 and 32 along with comparative data for oxidation ditch plants. These data show that the oxidation ditch plants perform better than conventional package plants.

Rotating Biological Contactors

Rotating biological contactors (RBC), as a secondary treatment alternative, are relatively new and only a few plants have been in operation for more than one year. Very little full scale data are available.

Recently, the data from the Gladstone, Michigan plant have become available affording a detailed analysis of the RBC process capability at one plant.

The Gladstone, Michigan plant is a 1 mgd plant and consists of primary sedimentation, RBC's designed for 1.94 gpd/sq ft of effective disc surface area, chemical addition, and final sedimentation. The plant started up in March of 1974 and reached stable operation by June of 1974. The manufacturer's literature would predict the following effluent quality based on the operating data when chemicals were not added. The actual results are in parenthesis.

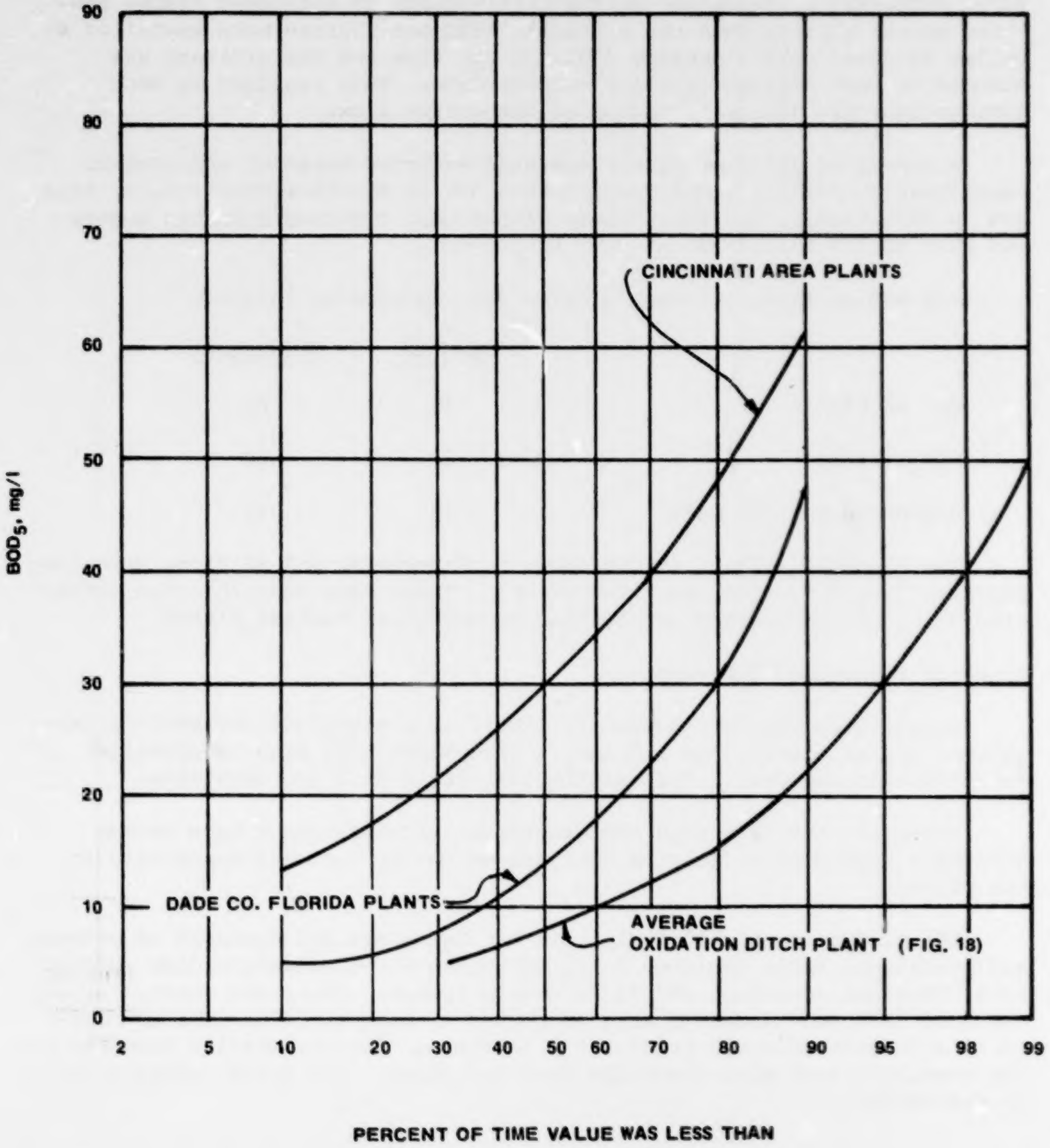


Figure 31. Activated sludge package plant reliability, BOD_5.

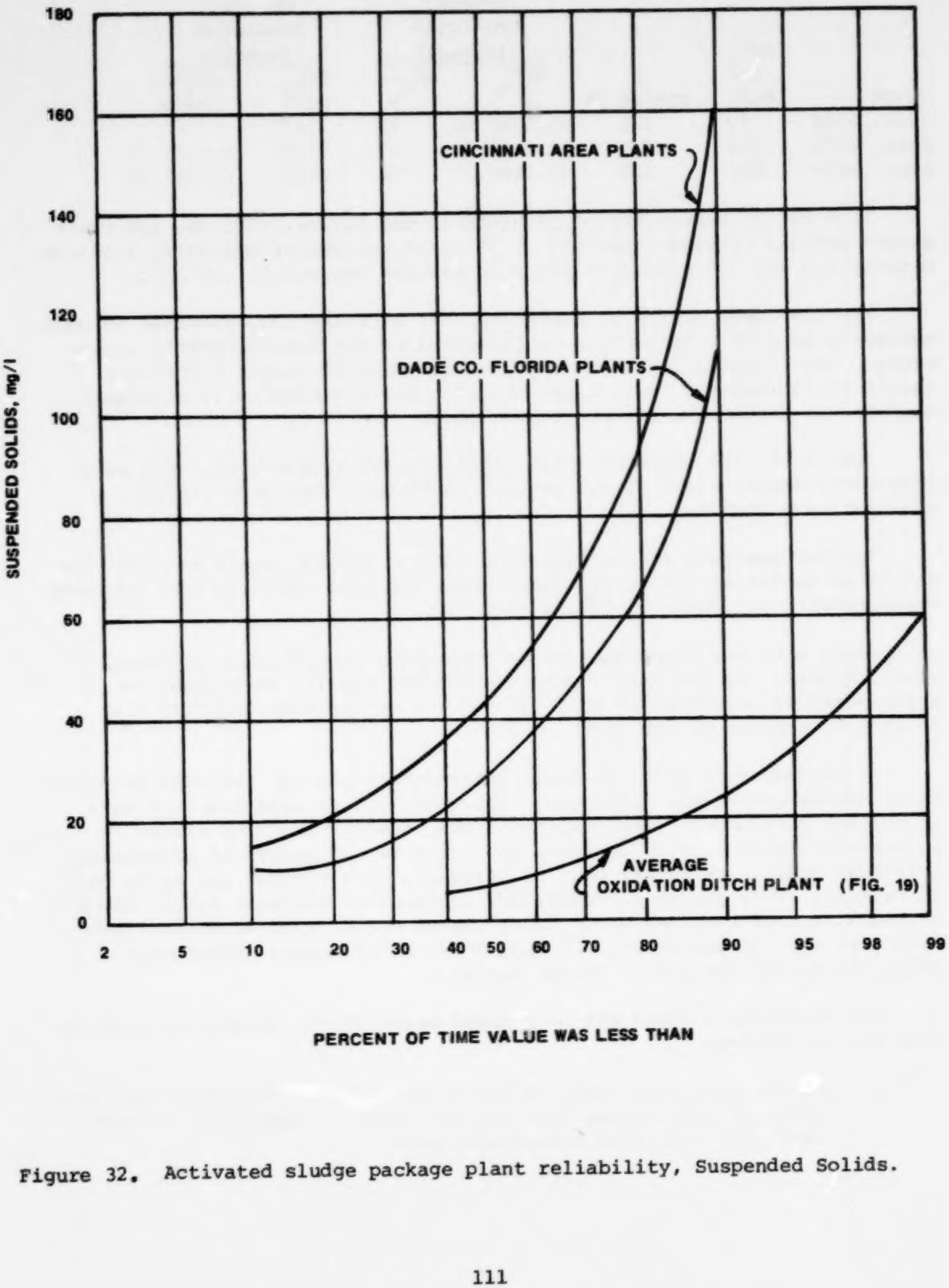


Figure 32. Activated sludge package plant reliability, Suspended Solids.

			Removal, Predicted (Actual)		Effluent Quality, Predicted (Actual)	
Month	BOD_5 in, mg/l	Q gpd/sq ft	BOD_5 %	NH_3-N %	BOD_5 mg/l	NH_3-N mg/l
June, 1974	99	1.5	92.9(82.8)	99	7(17)	-
July, 1974	105	1.7	92.4(81.9)	97	8(19)	0.6(<1.0)
Aug., 1974	102	1.5	92.2(88.2)	99	8(12)	0.2(<1.0)

Work by the University of Michigan at the Saline, Michigan RBC plant showed ammonia nitrogen removals of 95 to 98 percent at hydraulic loadings between 1.0 and 3.0 gpd/sq ft and temperatures between 11 and 21°C.

For the three months of operation, the effluent BOD_5 averaged 16 mg/l, whereas a BOD_5 of 8 mg/l would be predicted by the manufacturer's literature. The nitrification, according to the manufacturer's literature should be anticipated to be near complete, and the results (6 analyses) support the prediction for the months shown.

Therefore, the Gladstone plant BOD_5 results indicate that the manufacturer's design bases should be used cautiously when high quality effluents are required.

Further analysis of the Gladstone data on a daily basis was made for the three months of stable operation where chemical additions were not made. These results are shown on Figure 33.

These data are comparable to well operated conventional activated sludge plants. At low unit flows the effluent quality that might be anticipated is less than 20 mg/l, 50 percent of the time and less than 30 mg/l 90 percent of the time.

In September of 1974, chemical treatment of the RBC effluent prior to final clarification was initiated. About 80 mg/l of alum and 0. 5 mg/l of polymer were added. The results of the chemical addition proved to increase effluent quality as shown on Figure 34. A period of nonchemical treatment during the latter part of December, 1974 through the early part of February, 1975 showed a concomitant increase in effluent BOD_5. Ammonia nitrogen removal remained good during summer months when wastewater temperatures were in excess of 13°C, but lower nitrification rates were exhibited during the colder winter months.

The conclusions which may be reached based on the Gladstone, Michigan data are as follows:

1. At low unit flow rates (1.0-2.0 gpd/sq ft of effective disc area) effluent BOD_5 values from the RBC, will be comparable to conventional activated sludge processes.

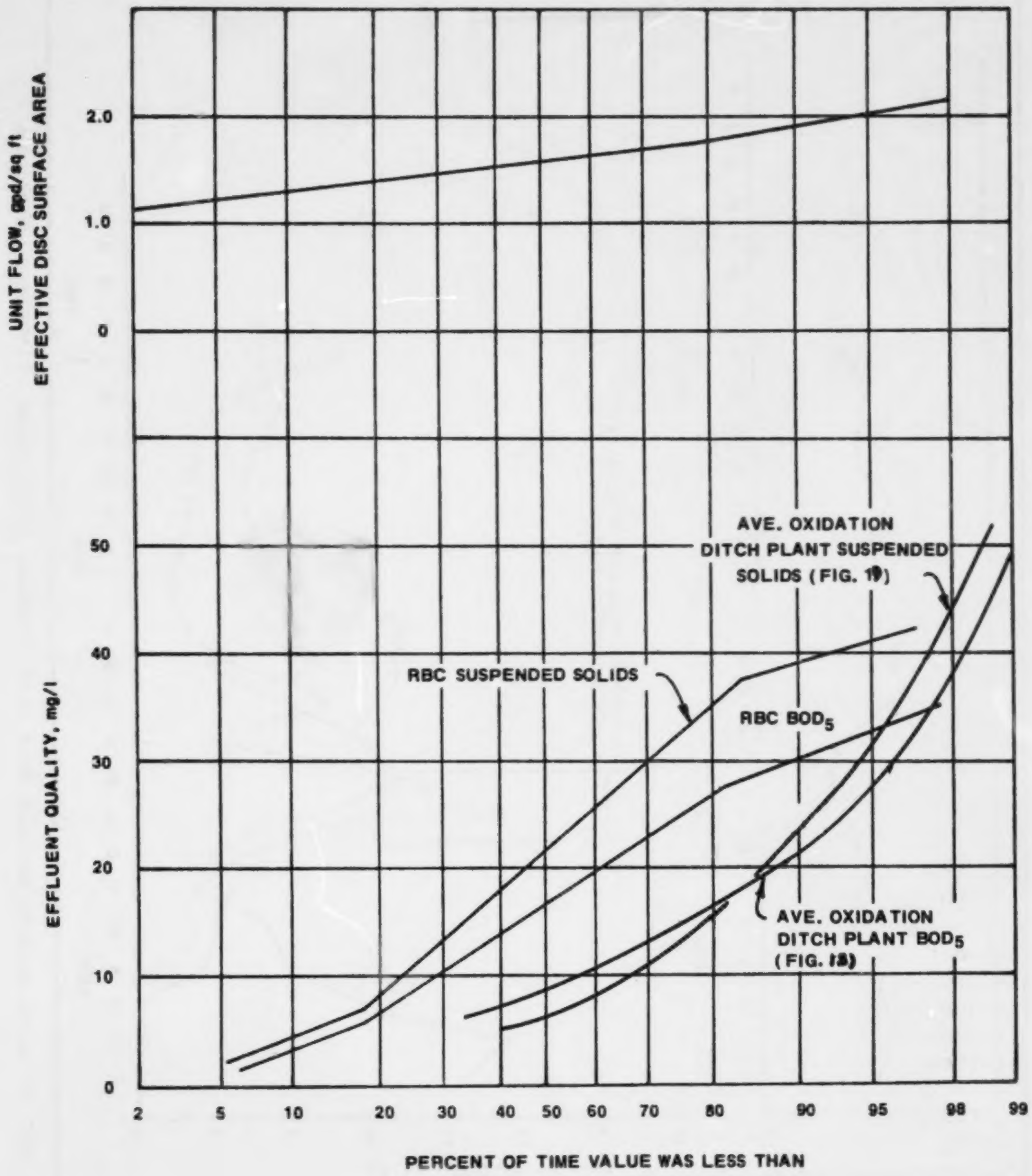


Figure 33. RBC effluent quality, Gladstone, Michigan.

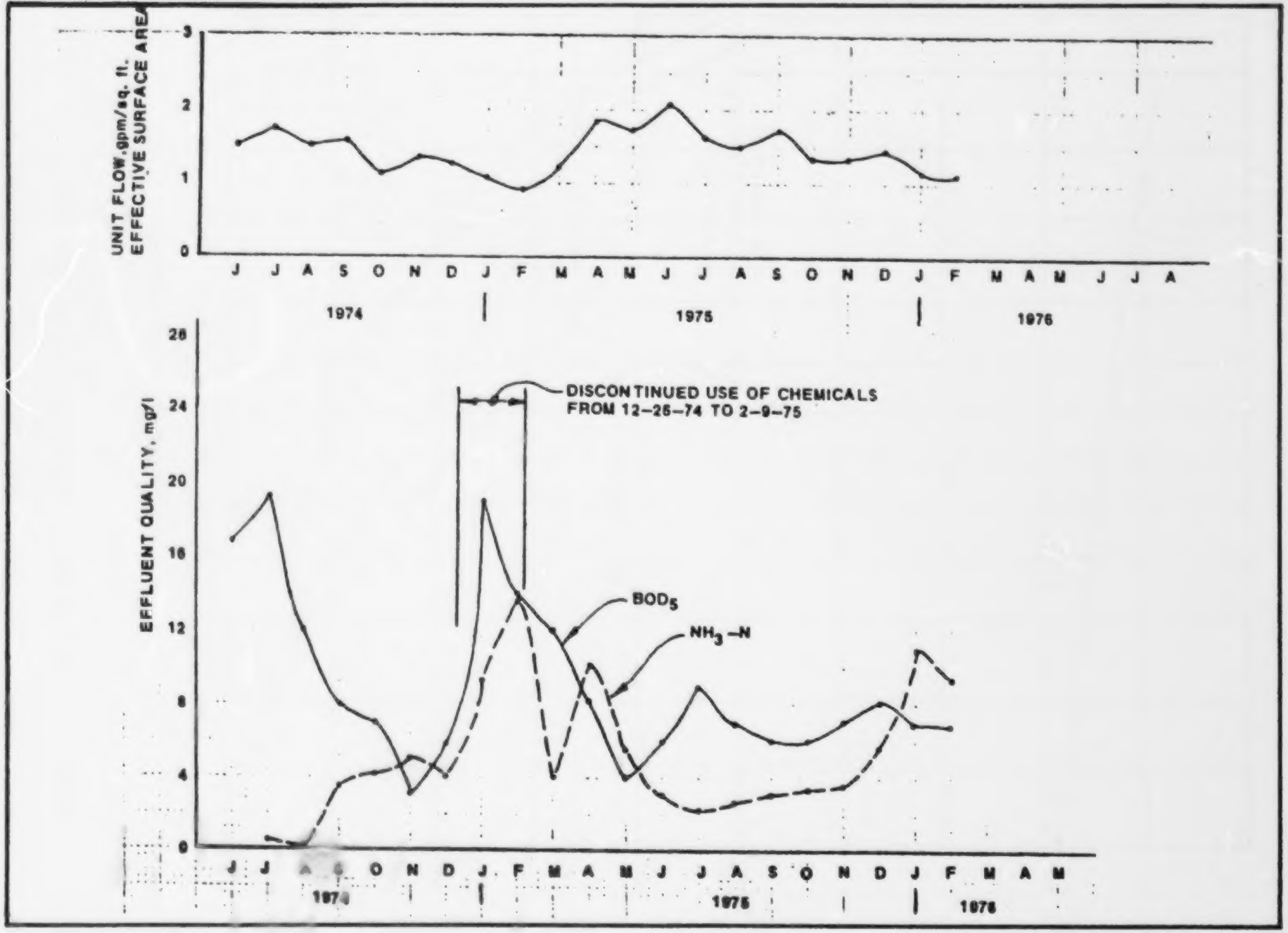


Figure 34. RBC effluent quality monthly data with chemical coagulation, Gladstone, Michigan.

2. With chemical coagulation additions, effluent BOD_5 values from the RBC will be consistently less than 10 mg/l, at the loadings used at Gladstone, Michigan.

3. Ammonia nitrogen concentrations in the Gladstone, Michigan effluent exceeded 2 mg/l consistently; however, good nitrification was experienced during the warmer summer months.

The RBC plant at Pewaukee, Wisconsin is a 0.46 mgd plant with a design disc hydraulic loading of 2.45 gpd/sq ft and final clarifier overflow rate of 500 gpd/sq ft. Average operating results without chemical addition for the years 1972, part of 1974, and 1976 are as follows.

	Final effluent, mg/l		Removal accross RBC, %	
Year	BOD_5	TSS	BOD_5	TSS
1972	20.3	15.8	84	83
1974 (9 months)	31.9	24.0	69	73
1976	25.0	30.0	86	88

The RBC plant at Edgewater, New Jersey operated at an average flow of 0.51 mgd, disc hydraulic loading of 2.5 gpd/sq ft, and final clarifier overflow rate of 518 gpd/sq ft produced the following results in 1973-1974.

Final effluent, mg/l		Removal across RBC, %	
BOD_5	TSS	BOD_5	TSS
37.6	21.6	75	69

PERFORMANCE SUMMARY

A review of effluent data for various biological waste treatment process indicates that realistic effluent BOD_5 and NH_3-N criteria for average to well operated plants may be assigned as shown in Table 23.

COSTS OF COMPETING BIOLOGICAL PROCESSES

Contact Stabilization

Aeration requirements are based on 30 cfm per thousand cubic feet of aeration volume and 20 cfm per thousand cubic feet of aerobic digester volume, plus any air lift pumps within the plant. Costs are based on centrifugal blowers with stand-by for the largest unit. All plants include internal chlorine contact and sludge stabilization chambers. The smaller sizes are of all steel construction, based on above-grade installation. Below-grade installation would be somewhat more expensive, due to the additional reinforcing required for external soil loads when the tanks are empty. Concrete pad costs were based on the diameter of the package plant plus 10 feet (5 feet extension beyond side of package plant). The larger sizes are based on steel internal equipment installed in poured reinforced concrete tankage. A contingency allowance of 15 percent was

TABLE 23. SUMMARY OF COMPETING PROCESS PERFORMANCE.

	BOD_5, mg/l		Ammonia-N,(*) mg/l
	50% of time	90% of time	50% of time
Trickling Filter (Average for various plant loadings).	25	40	2
Activated Sludge	15	40	1
Activated Sludge with Effluent Filtration	5	10	1
RBC	20	35	2
RBC With Chemical Coagulation	10	20	2
Activated Sludge Package Plants	20	50	1
Oxidation Ditch	8	21	1

*Assuming system is designed for nitrification at the lowest anticipated operation temperature for the location.

added to the manufacturer's estimate of the equipment and erection costs. In addition, electrical and instrumentation was calculated at 15 percent of equipment costs and contractors overhead and profit at 25 percent of equipment costs.

O & M costs were based on a labor rate of $9/hour including fringe benefits with seven day/week staffing as follows:

Capacity, mgd	Annual Labor, Hours
0.1	1,800
0.25	2,600
0.50	3,900
0.75	5,000
1.00	6,000
3.00	10,000

This amount of operator attendance is greater than many package plants have received in the past, but is the minimum felt by the authors to be consistent with satisfactory plant performance. Maintenance materials are based on 3% of the equipment costs.

Construction costs for contact stabilization plants are shown in Table 24. Operation and maintenance costs are shown in Table 25.

Extended Aeration

Air requirements are based on 2,100 cubic feet per pound of BOD removed (2 lb BOD/1,000 gallons/day). Positive displacement blowers with 100% standby are provided. As with contact stabilization plants, a contingency allowance of 15 percent was added to the manufacturer's estimate of the equipment and erection costs. In addition, electrical and instrumentation was calculated at 15 percent and contractors overhead and profit at 25 percent of equipment costs.

O & M costs were based on a labor rate of $9/hour including fringe benefits with seven day/week staffing as follows:

Capacity, mgd	Annual Labor, Hours
0.01	500
0.02	650
0.05	1,000
0.07	1,422
0.09	1,670
0.5	5,200

The amount of labor is based on CWC experiences with small package extended aeration plants and the labor required for proper O & M.

Construction costs for extended aeration package plants are shown in Table 26. Operation and maintenance costs are shown in Table 27.

TABLE 24. CONSTRUCTION COST OF CONTACT STABILIZATION PLANTS, 1976

	Construction cost, $1,000					
Plant capacity, mgd	0.1	0.25	0.50	0.75	1.0	3.0
Tankage & equipment in place	132	191	250	298	355	972
Chlorination equipment	6	6	6	10	12	33
Concrete work	7	14	24	36	50	137
Subtotal	145	211	280	344	417	1,142
Yardwork	20	30	40	48	58	158
Total construction cost	165	241	320	392	475	1,300

Costs do not include land, engineering, legal, or financing during construction, but include 25 percent contractors profit and overhead.

TABLE 25. OPERATION AND MAINTENANCE COSTS, CONTACT STABILIZATION PLANTS, 1976

	Annual cost, $1,000					
Plant capacity, mgd	0.1	0.25	0.50	0.75	1.0	3.0
Labor	16.2	23.4	35.1	45.0	54.0	90.0
Energy	2.4	5.7	11.4	18.0	24.0	65.0
Maintenance materials	4.0	6.0	7.5	9.0	10.6	15.0
Chlorine	.7	1.7	3.3	4.0	5.3	13.0
Total annual	23.3	36.8	57.3	76.0	93.9	183.0
Total, $/1,000 gal	0.638	0.403	0.314	0.278	0.257	0.167

Costs include labor at $9.00 per hour including fringe benefits and electrical energy at $0.03 per kwh.

TABLE 26. CONSTRUCTION COST OF EXTENDED AERATION PLANTS, 1976

	Construction cost, $1,000					
Plant capacity, mgd	0.01	0.02	0.05	0.07	0.09	0.5
Tankage & equipment in place	31	40	89	109	126	268
Chlorination equipment (cylinder mounted)	1	1	1	1	1	2
Concrete work	4	6	9	11	14	60
Subtotal	36	47	99	121	141	330
Yardwork	5	6	9	17	19	60
Total construction cost	41	53	108	138	160	390

Costs do not include land, engineering, legal, or financing during construction, but include 25 percent contractors profit and overhead.

TABLE 27. OPERATION AND MAINTENANCE COSTS, EXTENDED AERATION PLANTS, 1976

	Annual cost, $1,000					
Plant capacity, mgd	0.01	0.02	0.05	0.07	0.09	0.5
Labor	4.5	5.8	9.0	12.8	15.0	46.8
Energy	0.4	0.6	1.2	2.1	2.7	7.0
Maintenance materials	0.9	1.2	2.7	3.1	3.5	7.5
Chlorine	0.1	0.1	0.4	0.5	0.6	3.3
Total annual	5.9	7.7	13.3	18.5	21.8	64.6
Total $/1,000 gal	1.616	1.055	0.729	0.724	0.664	0.354

Costs include labor at $9.00 per hour including fringe benefits and electrical energy at $0.03 per kwh.

Conventional Activated Sludge

The oxidation ditch, extended aeration, and contact stabilization plant approaches provide an aerobically stabilized sludge for final disposal. In order to provide a comparable sludge, the conventional plants incorporate aerobic digesters. Sludge handling costs beyond aerobic stabilization are not included. Design of the aerobic digesters is based on criteria and procedures developed under EPA Contract 68-03-2186, Task 7 by CWC.

The activated sludge system schematic is shown in Figure 35. The major unit processes are primary sedimentation, activated sludge aeration, secondary sedimentation, aerobic digestion, and chlorination.

Using the primary effluent data and McKinney's complete mix model(1), the activated sludge system design criteria shown in Table 28 were developed. Unit process sizing is shown in Table 29. The aeration system design was limited to a peak hour oxygen uptake rate of 70 mg/l/hr. An SRT of 5 days was used. The return activated sludge pumps were sized for a 1 percent sludge concentration and completing a system solids balance. The secondary clarifiers were sized based on hydraulic overflow rate of 600 gpd/sq ft of average flow.

The chlorine contact basins are sized for a 30 minute detention time at peak dry weather flow (PDWF, 1.5 times design flow). A dosage rate of 10 mg/l was applied to the PDWF for sizing feed equipment.

General--

The construction and O & M costs for conventional activated sludge plants were developed by unit process through a review of the costs of actual plant construction and operation, equipment cost data from manufacturers, and published cost data. Generalized cost curves developed under EPA Contracts 68-03-2186 and 68-03-221 were used for estimating unit process costs. These generalized curves should not be used for comparing alternative processes as in this report. Individual plant costs must be developed based on the specific wastewater treatment plant design, local labor and material costs, and local climatic and site conditions.

Some of the limitations, in addition to the general local conditions discussed above, include no standby provisions, no specific modular sizing other than minimum available sizes, and no adjustments for local regulatory agency design restrictions.

Estimated construction costs are shown in Table 30 and O & M costs in Table 31.

Construction Costs--

Sedimentation--The source of the construction cost curve for sedimentation was the report to EPA, "Costs of Chemical Clarification of Wastewater", January, 1976, EPA Contract 68-03-2186. These cost data were developed from quantity takeoffs and equipment manufacturer's estimates.

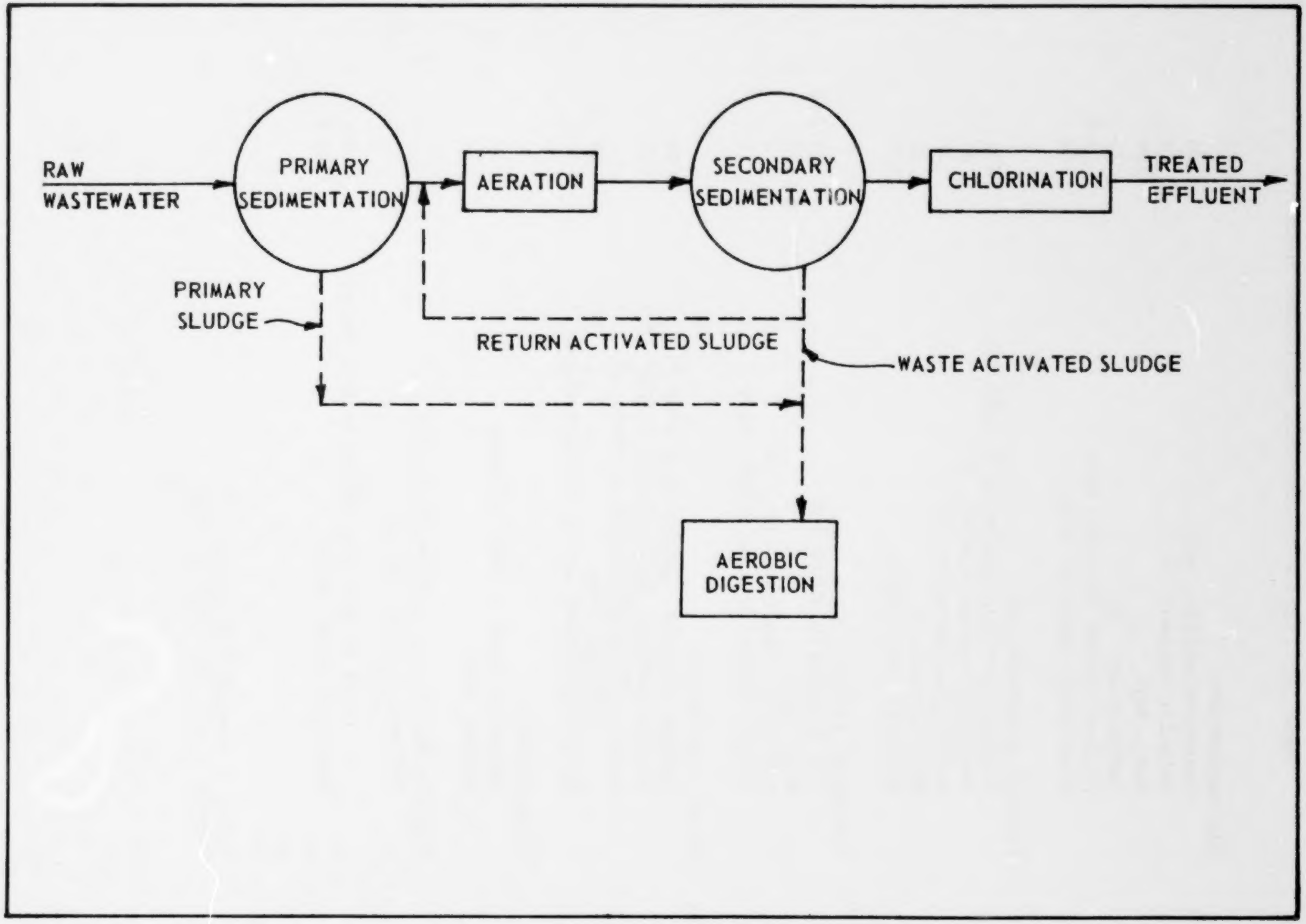

RAW
WASTEWATER
PRIMARY SEDIMENTATION
AERATION
SECONDARY SEDIMENTATION
CHLORINATION
TREATED
EFFLUENT
PRIMARY SLUDGE
RETURN ACTIVATED SLUDGE
WASTE ACTIVATED SLUDGE
AEROBIC DIGESTION

Figure 35. Activated sludge process schematic.

TABLE 28. DESIGN PARAMETERS, ACTIVATED SLUDGE

Raw wastewater:	
Suspended solids, mg/l	200
Volatile content, %	75
BOD_5, mg/l	200
Temperature, °C	20
Peaking factor (dry weather)	1.5
Alkalinity, mg/l as $CaCO_3$	200
Primary sedimentation:	
Surface loading, gpd/sq ft @ ADWF (*)	800
Suspended solids removal, %	65
Sludge concentration, %	5
BOD_5 removal, %	30
Effluent BOD_5, mg/l	140
Activated sludge:	
F/M lb BOD_5/lb MLVSS/day	0.355
MLSS, mg/l	3,150
Hydraulic detention time, hr	4
Solids retention time (SRT), days	5
Final clarifiers:	
Surface loading, gpd/sq ft @ ADWF(*)	600
Solids loading, lb/sq ft/day @ PDWF (**)	<35
Return activated sludge, percent of influent flow	46
Return activated sludge concentration, %	1
Chlorination:	
Detention time @ PDWF, minutes	30
Dosage, mg/l	10
Aerobic digestion, conventional air:	
Influent solids, percent	
1 mgd	1
5 and 10 mgd (automated decant cycle)	2
Detention time, days	15.6

*ADWF - Average dry weather flow
**PDWF - Peak dry weather flow

TABLE 29. UNIT PROCESS SIZES, ACTIVATED SLUDGE

Unit process or component	Plant capacity, mgd 1	 5	 10
Primary sedimentation tanks surface area, sq ft	1,250	6,250	12,500
Aeration tank volume, cu ft	22,300	111,500	223,000
Aerators, hp	40	200	400
Final clarifier surface area, sq ft	1,667	8,335	16,670
RAS pumps, mgd	.46/.69(*)	2.30/3.45	4.60/6.90
WAS pumps, gpm	45	225	450
Sludge pumps, gpm	15	75	150
Chlorination, cu ft	4,180	20,900	41,800
Chlorine feed equipment, tons/year (average/peak)	15.2/22.8(*)	76/114	152/228
Aerobic digestion			
Tank volume, cu ft	51,000	127,750	225,500
Aerators, hp	58	145	290

(*) Average/peak - Average flow is used to determine the power requirement and maintenance materials cost. Peak capacity is used to determine construction cost and labor requirement.

TABLE 30. CONSTRUCTION COSTS, ACTIVATED SLUDGE, 1976

	Construction cost, $1,000		
Plant capacity, mgd	1	5	10
Primary sedimentation tanks	70	260	440
Aeration basins	140	360	530
Aeration equipment	67	230	390
Secondary sedimentation tanks	90	330	590
Return activated sludge pumping sta.	70	160	230
Waste activated sludge pumping sta.	75	170	220
Primary sludge pumping station	44	93	140
Chlorine contact basins	51	140	200
Chlorination equipment	12	37	60
Aerobic digestion			
Basins	220	380	550
Aerators	78	160	280
Yardwork	128	325	508
Total construction cost	$1,045	$2,645	$4,138

Costs do not include land, engineering, legal, or financing during construction, but include 25 percent contractors profit and overhead.

TABLE 31. OPERATION AND MAINTENANCE COSTS, ACTIVATED SLUDGE, 1976

	Annual requirements						
	Labor		Energy				
Plant apacity	Hours	$1,000	1,000 KWH	$1,000	Maint. materials, $1,000	Chemicals, $1,000	Total annual, $1,000
1 mgd	5,498	49.5	547.8	16.4	11.7	3.3	80.9
5 mgd	10,020	90.2	1,874.6	56.2	24.7	16.7	187.8
10 mgd	14,340	129.0	3,653.2	109.6	36.1	33.4	308.1
ACTIVATED SLUDGE, SINGLE STAGE NITRIFICATION							
1 mgd							
10 mg/l NH_4-N	5,732	51.6	631.6	18.9	12.3	3.3	86.1
20 mg/l NH_4-N	5,832	52.5	701.6	21.1	12.5	3.3	89.4
30 mg/l NH_4-N	5,932	53.4	781.6	23.4	13.0	3.3	93.1
5 mgd							
10 mg/l NH_4-N	10,470	94.2	2,291.6	68.7	25.1	16.7	204.7
20 mg/l NH_4-N	10,970	98.7	2,591.6	77.7	26.2	16.7	219.3
30 mg/l NH_4-N	11,370	102.3	2,991.6	89.7	27.1	16.7	235.8
10 mgd							
10 mg/l NH_4-N	15,530	139.7	4,543.2	136.3	36.9	33.4	346.3
20 mg/l NH_4-N	16,320	146.9	5,243.2	157.3	37.9	33.4	375.5
30 mg/l NH_4-N	17,220	155.0	5,843.2	175.3	38.9	33.4	402.6
ACTIVATED SLUDGE, TWO STAGE NITRIFICATION							
1 mgd							
10 mg/l NH_4-N	7,038	63.3	635.1	19.1	14.1	3.3	99.8
20 mg/l NH_4-N	7,138	64.2	705.1	21.1	14.3	3.3	102.9
30 mg/l NH_4-N	7,238	65.1	785.1	23.6	14.8	3.3	106.8
5 mgd							
10 mg/l NH_4-N	12,820	115.4	2,337.9	70.1	28.6	16.7	230.8
20 mg/l NH_4-N	13,320	119.9	2,637.9	79.1	29.7	16.7	245.4
30 mg/l NH_4-N	13,720	123.4	3,037.9	91.1	30.6	16.7	261.8
10 mgd							
10 mg/l NH_4-N	18,940	170.5	4,629.8	139.9	43.4	33.4	386.2
20 mg/l NH_4-N	19,740	177.7	5,379.8	161.4	44.4	33.4	416.9
30 mg/l NH_4-N	20,640	185.8	5,929.8	177.9	45.4	33.4	442.5

Costs include labor at $9.00 per hour including fringe benefits and electrical energy at $0.03 per kwh.

Aeration Basins--Historical aeration basin cost data has been updated with results of other detailed cost studies by CWC and recent costs obtained from Black & Veatch and CH2M Hill.

These costs include excavation, concrete, walkways, in-basin process piping, handrails, and attendant costs. The construction cost data apply more closely to circular or square tanks used for complete mix activated sludge design than to long, narrow tanks as used for a plug flow mode.

Mechanical Aeration Equipment--Cost data for installed mechanical equipment have been derived from experienced cost data and equipment costs supplied by manufacturers.

Return Activated Sludge Pumping Station--The cost relationships for recycle pumping developed by Black and Veatch(29) were adjusted and used as a basis for estimating the cost of the return activated sludge pumping stations in this study. The costs shown by Black and Veatch have approximately doubled due to inflation, stricter OSHA requirements, and regulatory agency reliability standards.

The pumping stations are assumed to employ vertical diffusion vane pumping units with attendent valves, piping, and control facilities. The pump is suspended in the wet well and motors and motor control centers are housed in a superstructure.

Waste Sludge Pumping Stations--Waste sludge pumping equipment costs are based on the use of intermittent sludge pumping with positive displacement pumps. The cost data presented in the Black and Veatch cost curves were updated for this study.

Included in the pump station cost is an underground structure which houses the pumps and piping and is constructed adjacent to and in conjunction with the sedimentation basin. Also included is a superstructure which houses electrical control equipment.

Chlorination--Chlorine contact basin costs are based on the same construction used for the aeration basin costs. Chlorine feed equipment costs are based on chlorine gas feed and are taken from the draft report by CWC for the EPA "Estimating Initial Investment Costs and Operating and Maintenance Requirements of Stormwater Treatment Processes", EPA Contract 68-03-2186.

Operation and Maintenance Costs--

The operation and maintenance costs consist of labor, power and maintenance materials. The individual costs were developed through a variety of resources including recent CWC work for the EPA and the Black and Veatch study(29). In some instances, operating plants were consulted for information on labor requirements.

Sedimentation Basins--O & M requirements are based on the Black and Veatch report(29).

Mechanical Aeration--Operation and maintenance requirements for the aeration systems are expressed in terms of the installed aerator horsepower. Labor requirements are based on the Black and Veatch report(29). The power requirements were calculated on the basis of an assumed oxygen transfer of 2.0 lb O_2/hp-hr or 3.0 lb O_2/KWH. Maintenance material costs are based on the Black and Veatch report(29).

Return Activated Sludge Pumping Station--The return activated sludge pumping station labor requirements are based on the Black and Veatch report(29) The power requirements are based on the Black and Veatch report(29) assuming a head of 10 feet. The maintenance material costs are an update of the Black and Veatch report(29).

Waste Sludge Pumping Station--Labor requirements for the waste sludge pumping stations are based on the Black and Veatch report(29). The power requirements were based on a pumping head of 25 feet and a pumping efficiency of 40 percent (progressing cavity pumps). Maintenance material costs were updated from the black and Veatch report(29).

Chlorination--Labor requirements and maintenance material costs for chlorination are based on the Black and Veatch report(29). The chlorine costs are based on recent quotes for ton cylinders and tank car lots.

BIOLOGICAL NITRIFICATION

Single Stage Nitrification

The design of this system is based on a mean cell residence time of 10 days to achieve nitrification in a single stage activated sludge system. A complete discussion of nitrification is contained in Section 4. Tables 32 and 33 show design parameters for the nitrification system and the resulting unit process sizes. Construction and O & M costs are based on the same sources described for the conventional activated sludge system. The effects of different average ammonium nitrogen concentrations (10, 20, and 30 mg/l) were estimated based upon providing 4.6 lb oxygen per lb of ammonium nitrogen. Peak hourly ammonium nitrogen concentrations of twice the average were assumed with peak hourly BOD concentrations of 1.5 times the average. An oxygen transfer efficiency of 2 lb /hp-hr was used.

Estimated construction costs are shown in Table 34 and O & M costs in Table 31.

Two Stage Nitrification

Tables 35, 36, and 37 show the design parameters used for this system and resulting costs. The same basic assumptions used for the single stage nitrification system on ammonium concentrations, oxygen transfer, and similar parameters were used for two stage.

TABLE 32. DESIGN PARAMETERS, SINGLE STAGE ACTIVATED SLUDGE NITRIFICATION

Raw wastewater:	
Suspended solids, mg/l	200
Volatile content, %	75
BOD_5, mg/l	200
Temperature, °C	20
Peaking factor (dry weather)	1.5
Alkalinity, mg/l as $CaCO_3$	200
Primary sedimentation:	
Surface loading, gpd/sq ft @ ADWF(*)	800
Suspended solids removal, %	65
Sludge concentration, %	5
BOD_5 removal, %	30
Effluent BOD_5, mg/l	140
Activated sludge:	
F/M, lb BOD_5/lb MLVSS/day	0.20
MLSS, mg/l	3,270
Hydraulic detention time, hr	7
Solids retention time (SRT), days	10
Ammonium concentrations, average	10,20,30 mg/l
Peak hourly ammonium concentration	2 x average
Peak hourly BOD_5	1.5 x average
Final clarifiers:	
Surface loading, gpd/sq ft @ ADWF(*)	600
Solids loading, lb/sq ft/day @ PDWF(**)	35
Return activated sludge, percent of influent flow	69
Return activated sludge concentration, percent	0.8
Chlorination:	
Detention time @ PDWF, minutes	30
Dosage, mg/l	10
Aerobic digestion, conventional air:	
Influent solids, percent	
1 mgd	1
5 and 10 mgd (automated decant cycle)	2
Detention time, days	15.6

(*) ADWF - Average dry weather flow

(**) PDWF - Peak dry weather flow

TABLE 33. UNIT PROCESS SIZES, SINGLE STAGE ACTIVATED SLUDGE NITRIFICATION

Unit process or component	Plant capacity, mgd		
	1	5	10
Primary sedimentation tanks surface area, sq ft	1,250	6,250	12,500
Aeration tank volume, cu ft	39,000	195,000	390,000
Aerators, hp			
10 mg/l NH_4-N	50	270	540
20 mg/l NH_4-N	70	350	700
30 mg/l NH_4-N	85	425	850
Final clarifier surface area, sq ft	1,667	8,335	16,670
RAS pumps, mgd	.69/1.03(*)	3.45/5.18	6.90/10.3
WAS pumps, gpm	36	180	360
Sludge pumps, gpm	15	75	150
Chlorination contact tank volume, cu ft	4,180	20,900	41,800
Chlorination feed equipment, tons/year (average/peak)	15.2/22.8(*)	76/114	152/228
Aerobic digestion			
Tank volume, cu ft	51,100	127,750	255,500
Aerators, hp	58	145	290

(*) Average/Peak - Average flow is used to determine the power requirement and maintenance materials cost. Peak capacity is used to determine construction cost and labor requirement.

TABLE 34. CONSTRUCTION COST, SINGLE STAGE ACTIVATED SLUDGE NITRIFICATION

	Construction cost, $1,000		
Plant capacity, mgd	1	5	10
Primary sedimentation tanks	70	260	440
Aeration basins	190	570	850
Aeration equipment(*)	130	450	800
Secondary sedimentation tanks	90	330	590
Return activated sludge pumping sta.	86	200	300
Waste activated sludge pumping sta.	90	190	270
Primary sludge pumping station	44	93	140
Chlorine contact basins	51	140	200
Chlorination equipment	12	37	60
Aerobic digestion			
Basins	220	380	550
Aerators	78	160	280
Yardwork	149	393	627
Total construction cost			
at 20 mg/l NH_4-N	$1,210	$3,203	$5,107
at 10 mg/l NH_4-N	$1,185	$3,023	$4,957
at 30 mg/l NH_4-N	$1,240	$3,283	$5,207

(*) Table based on 20 mg/l NH_4-N. Construction costs of aeration equipment at:

10 mg/l NH_4-N: 1 mgd = $105,000; 5 mgd = $270,000; 10 mgd = $550,000
20 mg/l NH_4-N: 1 mgd = $160,000; 5 mgd = $530,000; 10 mgd = $900,000

The costs do not include land, engineering, legal, or financing during construction, but include 25 percent contractor profit and overhead.

TABLE 35. DESIGN PARAMETERS, TWO STAGE ACTIVATED SLUDGE NITRIFICATION

Raw wastewater:	
Suspended solids, mg/l	200
Volatile content, %	75
BOD$_5$, mg/l	200
Temperature, ^{o}C	20
Peaking factor (dry weather)	1.5
Alkalinity, mg/l as CaCO$_3$	200
Primary sedimentation:	
Surface loading, gpd/sq ft @ ADWF(*)	800
Suspended solids removal, %	65
Sludge concentration, %	5
BOD$_5$ removal, %	30
Effluent BOD$_5$, mg/l	140
Activated sludge:	
First stage complete mix aeration	
F/M, lb BOD$_5$/lb MLVSS/day	0.34
MLSS, mg/l	3,000
Hydraulic detention time, hr	3
Solids retention time (SRT), days	4
Second stage plug flow aeration	
F/M, lb BOD$_5$/lb MLVSS/day	0.12
MLSS, mg/l	2,400
Hydraulic detention time, hr	4
Solids retention time (SRT), days	10
Ammonium concentrations, average	10,20,30, mg/l
Peak hourly ammonium concentration	2 x average
Peak hourly BOD$_5$	1.5 x average
Final clarifiers - first & second stage:	
Surface loading, gpd/sq ft @ ADWF(*)	600
Solids loading, lb/sq ft/day @ PDWF(**)	35
Return activated sludge, percent of influent flow	69
Return activated sludge concentration, percent	0.8
Chlorination:	
Detention time @ PDWF, minutes	30
Dosage, mg/l	10
Aerobic digestion, conventional air:	
Influent solids, percent	
1 mgd	1
5 and 10 mgd (automated decant cycle)	2
Detention time, days	15.6

(*) ADWF - Average dry weather flow
(**) PDWF - Peak dry weather flow

TABLE 36. UNIT PROCESS SIZES, TWO STAGE ACTIVATED SLUDGE NITRIFICATION

Unit process or component	Plant capacity, mgd 1	 5	 10
Primary sedimentation tanks surface area, sq ft	1,250	6,250	12,500
First stage aeration tank			
Volume, cu ft	16,700	83,400	167,000
Aerators, hp	80	400	800
Second stage aeration tank			
Volume, cu ft	22,300	111,500	223,000
Aerators, hp			
10 mg/l NH_4-N	15	75	150
20 mg/l NH_4-N	30	150	300
30 mg/l NH_4-N	50	250	500
Final clarifier surface area, sq ft	1,667	8,335	16,670
RAS pumps, mgd	.69/1.03(*)	3.45/5.18	6.90/10.3
WAS pumps, gpm	36	180	360
Sludge pumps, gpm	15	75	150
Chlorination contact tank volume, cu ft	4,180	20,900	41,800
Chlorination feed equipment, tons/year (average/peak)	15.2/22.8(*)	76/114	152/228
Aerobic digestion			
Tank volume, cu ft	51,100	127,750	255,500
Aerators, hp	58	145	290

(*) Average/Peak - Average flow is used to determine the power requirement and maintenance materials cost. Peak capacity is used to determine construction cost and labor requirement.

TABLE 37. CONSTRUCTION COST, TWO STAGE NITRIFICATION, 1976

	Construction cost, $1,000		
Plant capacity, mgd	1	5	10
Primary sedimentation tanks	70	260	440
Aeration basins, first stage(*)	140	360	530
Aeration basins, second stage(*)	130	340	480
Aeration equipment, first stage	67	230	390
Aeration equipment, second stage	63	210	360
Secondary sedimentation tanks	90	330	590
Nitrif. sedimentation tanks	90	330	590
Return activated sludge pumping sta.	70	160	230
Nitrif. RAS station	70	160	230
Waste activated sludge pumping sta.	75	170	220
Primary sludge pumping station	44	93	140
Chlorine contact basins	51	140	200
Chlorination equipment	12	37	60
Aerobic digestion			
Basins	220	380	550
Aerators	78	160	280
Yardwork	178	470	741
Total construction cost			
at 20 mg/l NH_4-N	$1,448	$3,830	$6,031
at 10 mg/l NH_4-N	1,413	3,730	5,821
at 30 mg/l NH_4-N	1,490	3,942	6,171

*Table based on 20 mg/l NH_4-N. Construction costs of both stages of aeration equipment at:
10 mg/l NH_4-N: 1 mgd = $235,000; 5 mgd = $600,000; 10 mgd = $800,000;
20 mg/l NH_4-N: 1 mgd = $312,000; 5 mgd = $812,000; 10 mgd = $1,150,000

These costs do not include land, engineering, legal, or financing during construction, but include 25 percent contractor profit and overhead.

BIOLOGICAL DENITRIFICATION

Mixed Reactor

Tables 38 and 39 show the design basis for denitrification in a mixed, uncovered reactor and the unit process sizing. The anoxic denitrification reactor is followed by an aerobic stabilization reactor for removal of any excess methanol. Solids are then removed in a clarifier and recycled to the denitrification reactor. Estimated construction costs are shown in Table 40 and O & M costs in Table 41.

Fixed Film Denitrification

The costs of fixed film denitrification were estimated based upon the following criteria: 6 ft deep bed, gravity system, 2-4 mm sand, 2.7 gpm/sq ft at average flow, backwash 15 min/day at 25 gpm/sq ft and 25 ft TDH with auxiliary air scour, 3:1 methanol to NO_3-N ratio. Costs for the fixed film system were based on the work conducted on filtration system costs by CWC under EPA Contract 68-03-2186.

Construction costs are shown in Table 42 and operation and maintenance costs in Table 43.

PHYSICAL-CHEMICAL NITROGEN REMOVAL

Cost information was developed for the physical-chemical alternatives of breakpoint chlorination, selective ion exchange, and ammonia stripping. Construction costs are shown in Table 44 and operation and maintenance costs in Table 45.

Breakpoint Chlorination

The basic design criteria used were as follows:

Provide 30 seconds of rapid mixing, G = 900

Peak NH_3 concentration = 2 x average NH_4 concentration

Chlorine feed capacity = 10 x peak NH_4-N concentration @ average flow

Costs of chlorine contact facilities were not included because such facilities would normally be provided for disinfection purposes even without the need to remove nitrogen.

Costs were estimated for average NH_4-N concentrations of 10, 20 and 30 mg/l. Construction and O & M cost information was derived from reports prepared under EPA Contract 68-03-2186 by CWC. Chlorine usage in the various size facilities is as follows:

TABLE 38. DESIGN PARAMETERS, MIXED REACTOR DENITRIFICATION

Denitrification reactor:	
Type	Suspended growth
Nitrate removal rate, lb NO_3-N/lb MLVSS/day	0.1
MLVSS, mg/l	1500
Mixer type	Submerged turbine
Mixer size, hp/1,000 cu ft	0.5
Aerated stabilization reactor:	
Detention, minutes	50
Aeration, hp/1,000 cu ft	1
Final clarification:	
Overflow rate, gpd/sq ft	700
Sludge recycle, %	50
Methanol feed:	
Methanol: nitrogen ratio	3:1
NO_3-N concentrations, mg/l	10, 20, 30

TABLE 39. UNIT PROCESS SIZES, MIXED REACTOR DENITRIFICATION

Unit process or component	Plant capacity, mgd 1	 5	 10
Denitrification reactor			
Volume, cu ft/mixing, hp			
10 mg/l NO_3-N	8,900/5	44,500/22.5	89,000/45
20 mg/l NO_3-N	17,800/10	89,000/45	178,000/90
30 mg/l NO_3-N	26,700/15	133,500/67.5	267,000/135
Aerated stabilization reactor			
Volume, cu ft	4,500	22,500	45,000
Aeration, hp	5	22.5	45
Clarifier			
Area, sq ft	1,500	7,500	15,000
Return sludge, mgd	0.5/0.75*	2.5/3.75	5/7.5
Methanol feed, lbs/hr			
10 mg/l NO_3-N	9	44	87
20 mg/l NO_3-N	17	87	174
30 mg/l NO_3-N	26	131	261

* Average/Peak - Average flow is used to determine the power requirements and maintenance materials cost. Peak capacity is used to determine construction cost and labor requirement.

TABLE 40. CONSTRUCTION COSTS, MIXED REACTOR DENITRIFICATION, 1976

	Construction costs, $1,000								
	1 mgd plant			5 mgd plant			10 mgd plant		
	10 mg/1 NO_3-N	20 mg/1 NO_3-N	30 mg/1 NO_3-N	10 mg/1 NO_3-N	20 mg/1 NO_3-N	30 mg/1 NO_3-N	10 mg/1 NO_3-N	20 mg/1 NO_3-N	30 mg/1 NO_3-N
Denitrification reactor									
Basin	100	160	180	250	360	440	360	510	680
Mixers	16	27	40	50	90	120	90	160	200
Aerated stabilization reactor									
Basin	60	60	60	160	160	160	250	250	250
Aerators	16	16	16	50	50	50	90	90	90
Clarifier	90	90	90	290	290	290	600	600	600
Return sludge	70	70	70	160	160	160	250	250	250
Methanol feed & storage	35	50	65	50	80	120	80	150	200
Subtotal	387	473	521	1010	1190	1340	1720	2010	2270
Yardwork	54	66	73	141	167	188	241	281	318
Total construction cost	441	539	594	1151	1357	1528	1961	2291	2588

These costs are for the denitrification process only and do not include land, engineering, legal, or financing during construction, but include 25 percent contractor profit and overhead.

TABLE 41. OPERATION AND MAINTENANCE COSTS, MIXED REACTOR DENITRIFICATION, 1976

	Annual requirements						
	Labor		Energy				
Plant capacity	Hours	$1,000	1,000 KWH	$1,000	Maint. materials, $1,000	Chemicals, $1,000	Total annual, $1,000
1 mgd							
10 mg/1 NO_3-N	3,400	30.6	65.8	2.0	4.3	6.9	43.8
20 mg/1 NO_3-N	3,650	32.9	91.1	2.7	4.8	13.7	54.1
30 mg/1 NO_3-N	3,900	35.1	116.3	3.5	5.1	20.5	64.2
5 mgd							
10 mg/1 NO_3-N	5,070	45.6	301.0	9.0	7.9	34.3	96.8
20 mg/1 NO_3-N	5,600	50.4	426.3	12.8	8.9	68.5	140.6
30 mg/1 NO_3-N	6,150	55.4	552.0	16.6	9.9	102.7	184.6
10 mgd							
10 mg/1 NO_3-N	6,800	61.2	586.3	17.6	11.3	68.5	158.6
20 mg/1 NO_3-N	7,700	69.3	837.4	25.1	13.2	136.9	244.5
30 mg/1 NO_3-N	8,200	73.8	1,088.1	32.6	14.6	205.4	326.4

These costs are for the denitrification process only and include labor at $9.00 per hour including fringe benefits and electrical energy at $0.03 per kwh.

TABLE 42. CONSTRUCTION COSTS, FIXED FILM DENITRIFICATION, 1976

	Construction cost, $1,000		
Plant capacity, mgd	1	5	10
Structure	336	776	1,506
Media	12	40	80
Air/water backwash	160	150	280
Methanol feed & storage	50	80	150
Yardwork	78	146	282
Total construction cost			
at 20 mg/l NO_3-N	$636	$1,192	$2,298
at 10 mg/l NO_3-N	$619	$1,138	$2,218
at 30 mg/l NO_3-N	$653	$1,238	$2,355

These costs do not include land, engineering, legal, or financing during construction, but include 25 percent contractor profit and overhead.

TABLE 43. OPERATION AND MAINTENANCE COSTS, FIXED FILM DENITRIFICATION, 1976

	Annual requirements						
	Labor		Energy				
Plant capacity	Hours	$1,000	1,000 KWH	$1,000	Maint. materials, $1,000	Chemicals, $1,000	Total annual, $1,000
1 mgd							
10 mg/1 NO_3-N	3,850	34.6	11.5	0.3	1.2	6.9	43.1
20 mg/1 NO_3-N	4,000	36.0	11.8	0.4	1.2	13.7	51.3
30 mg/1 NO_3-N	4,150	37.4	12.0	0.4	1.3	20.5	59.6
5 mgd							
10 mg/1 NO_3-N	4,500	40.5	43.0	1.3	2.1	34.3	78.2
20 mg/1 NO_3-N	4,800	43.2	43.8	1.3	2.3	68.5	115.3
30 mg/1 NO_3-N	5,100	45.9	44.5	1.3	2.6	102.7	152.5
10 mgd							
10 mg/1 NO_3-N	5,400	48.6	73.0	2.2	3.1	68.5	122.4
20 mg/1 NO_3-N	5,800	52.2	74.1	2.2	3.6	137.0	195.0
30 mg/1 NO_3-N	6,200	55.8	74.8	2.3	4.1	205.4	267.6

These costs include labor at $9.00 per hour including fringe benefits and electrical energy at $0.03 per kwh.

TABLE 44. CONSTRUCTION COSTS, PHYSICAL-CHEMICAL NITROGEN REMOVAL, 1976

	Construction cost, \$1,000		
	NH_4-N concentration		
Process & plant capacity, mgd	10 mg/l	20 mg/l	30 mg/l
Breakpoint chlorination			
0.01	4.6	7.1	12.3
0.1	17.2	25.4	36.1
1.0	70.5	114.8	176.2
5.0	229.5	377.1	623.0
10.0	377.1	696.7	1,475.4
Selective ion exchange			
0.01	98.4	98.4	98.4
0.1	147.5	147.5	147.5
1.0	442.6	442.6	442.6
5.0	1,557.4	1,557.4	1,557.4
10.0	2,704.9	2,704.9	2,704.9
Ammonia stripping			
0.01	3.9	3.9	3.9
0.1	31.1	31.1	31.1
1.0	245.9	245.9	245.9
5.0	1,065.6	1,065.6	1,065.6
10.0	1,967.2	1,967.2	1,967.2

Costs do not include land, engineering, legal, or financing during construction, but include 25 percent contractor overhead and profit.

TABLE 45. OPERATION AND MAINTENANCE COSTS, PHYSICAL-CHEMICAL NITROGEN REMOVAL, 1976

	Annual cost, $1,000		
	NH_4-N concentration		
Process & plant capacity, mgd	10 mg/l	20 mg/l	30 mg/l
Breakpoint chlorination			
0.01	3.7	4.7	5.4
0.1	11.0	17.0	23.5
1.0	52.0	100.0	140.0
5.0	170.0	280.0	400.0
10.0	210.0	380.0	600.0
Selective ion exchange			
0.01	13.0	14.0	15.0
0.1	18.0	19.5	21.0
1.0	40.0	47.0	55.0
5.0	130.0	150.0	180.0
10.0	210.0	250.0	320.0
Ammonia stripping			
0.01	2.8	2.8	2.8
0.1	6.2	6.2	6.2
1.0	18.0	18.0	18.0
5.0	57.0	57.0	57.0
10.0	170.0	170.0	170.0

Costs include labor at $9.00 per hour including fringe benefits and electrical energy at $0.03 per kwh.

	10 mg/l NH_4-N	20 mg/l NH_4-N	30 mg/l NH_4-N
0.01 mgd	1.5 ton/yr	3.1	4.5
0.1	15	31	45
1	150	310	450
10	1500	3100	4500

Costs of chlorine were based on use of ton cylinders for quantities up to 450 tons/year ($0.11/lb) and on tank cars over 450 tons/year. Between 1 and 10 mgd, the demurrage cost on the rail cars will result in an effective chlorine cost in excess of $0.05/lb and a gradually decreasing cost down to $0.05/lb at 10 mgd. To determine O & M costs, a labor rate of $9.00 per hour and energy cost of $0.03 per kwh were used. The same unit costs for labor and power were used for all alternatives.

Selective Ion Exchange

The costs for this process are based on use of clinoptilolite exchange media in gravity structures with recovery of the regenerant in closed-loop stripping towers. A minimum of four exchangers was provided for each capacity. A four foot deep clinoptilolite bed loaded at 5.25 gpm/sq ft was used. Exchanger construction costs are based on cost of gravity filtration structures developed by CWC under EPA Contract 68-03-2186. Costs include the exchange structure, backwash facility, influent pumping, clarification-softening facility for the spent regenerant, and closed-loop stripping tower for regenerant recovery. Construction costs are essentially unaffected by ammonia concentration but regeneration frequency and operating costs increase as ammonia concentration increases. Costs for the closed-loop tower modules (such as illustrated on pages 9-75 and 9-76 of EPA's Technology Transfer Manual on Nitrogen Control) are based on the estimated cost of such units for the Upper Occoquan, Virginia plant. Influent pumping costs are based on 15 ft TDH with regenerant recovery pumping at 35 ft TDH. Chemical costs are based on those projected for the Upper Occoquan plant.

Ammonia Stripping

Construction costs are based on a tower loading rate of 1 gpm/sq ft with a tower packing of the type used in the Orange County, California plant (see page 9-90, EPA's Technology Transfer Manual on Nitrogen Control) with 24-foot packing depth. Construction and O & M costs include influent pumping (50 ft TDH). The costs do not include elevating the pH of the wastewater to an adequate level for stripping nor of subsequent downward pH adjustment following stripping. The costs of the stripping process to provide a given percentage removal of ammonium-nitrogen are independent of influent concentration (at a given temperature).

SUMMARY AND COMPARISON

A complete discussion of the data developed herein is contained in Section 8 based on the following comparisons of competing unit processes.

Biological Treatment Processes

Comparative construction, operation and maintenance, and total annual costs for the various biological treatment processes considered is shown in Figures 36, 37, and 38. The total annual costs are the O & M costs plus the amortized construction costs at 7 percent over 20 years. A summary of operation and maintenance costs from a number of operating plants is shown in Table 46 for comparative purposes. This information was summarized from EPA Region inspection and technical assistance reports.

A summary of biological treatment process characteristics is shown in Table 47. Some of the factors are difficult to define specifically, therefore, they are covered by general ranges.

The relative performance of various biological treatment processes is illustrated in Figures 26 through 34. These data will allow some general comparisons between the various biological treatment processes in use.

Nitrification Processes

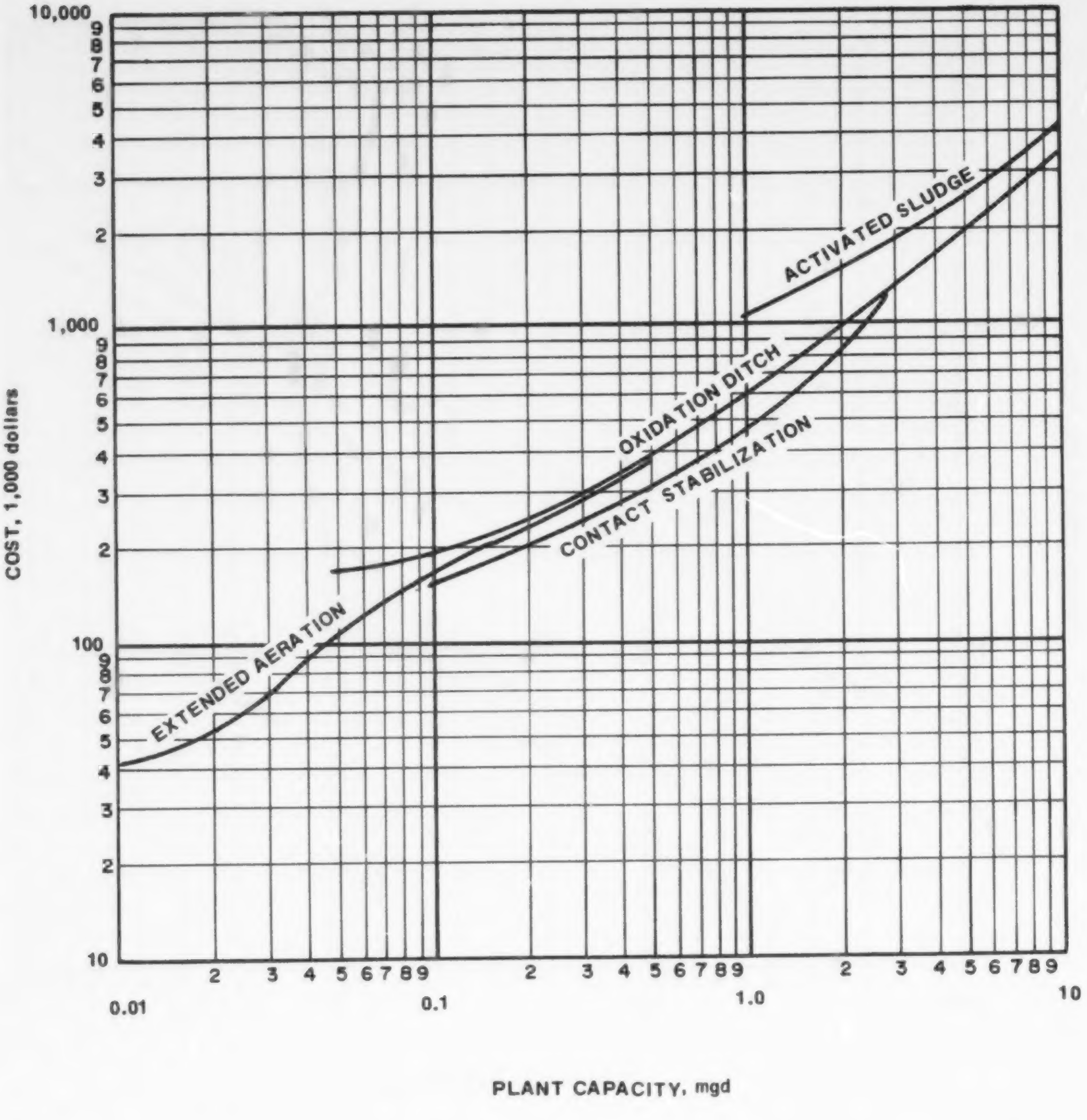
A comparison of the incremental construction and O & M costs (over the basic biological treatment process) is shown in Figures 39 and 40. All processes will provide essentially complete nitrification with proper design.

Nitrogen Removal Processes

Comparable incremental construction, operation and maintenance, and total annual costs for the various biological and physical chemical nitrogen removal processes are shown in Figures 41, 42, and 43. The total annual costs are the incremental O & M costs plus the incremental amortized construction costs at 7 percent over 20 years.

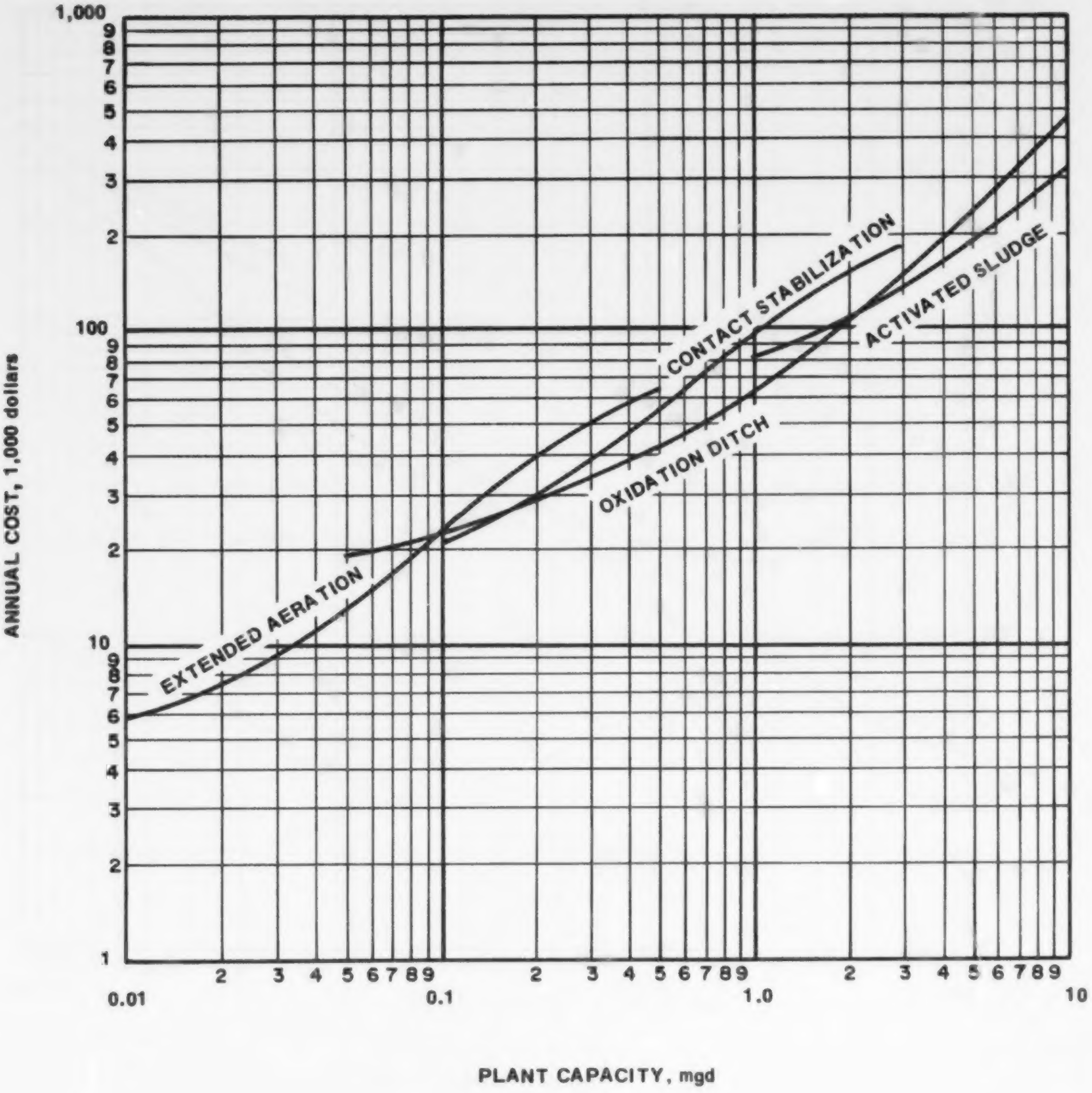
The biological nitrogen removal processes include costs for the required nitrification and denitrification steps.

Some comparative characteristics of the nitrogen removal processes are shown in Table 48.



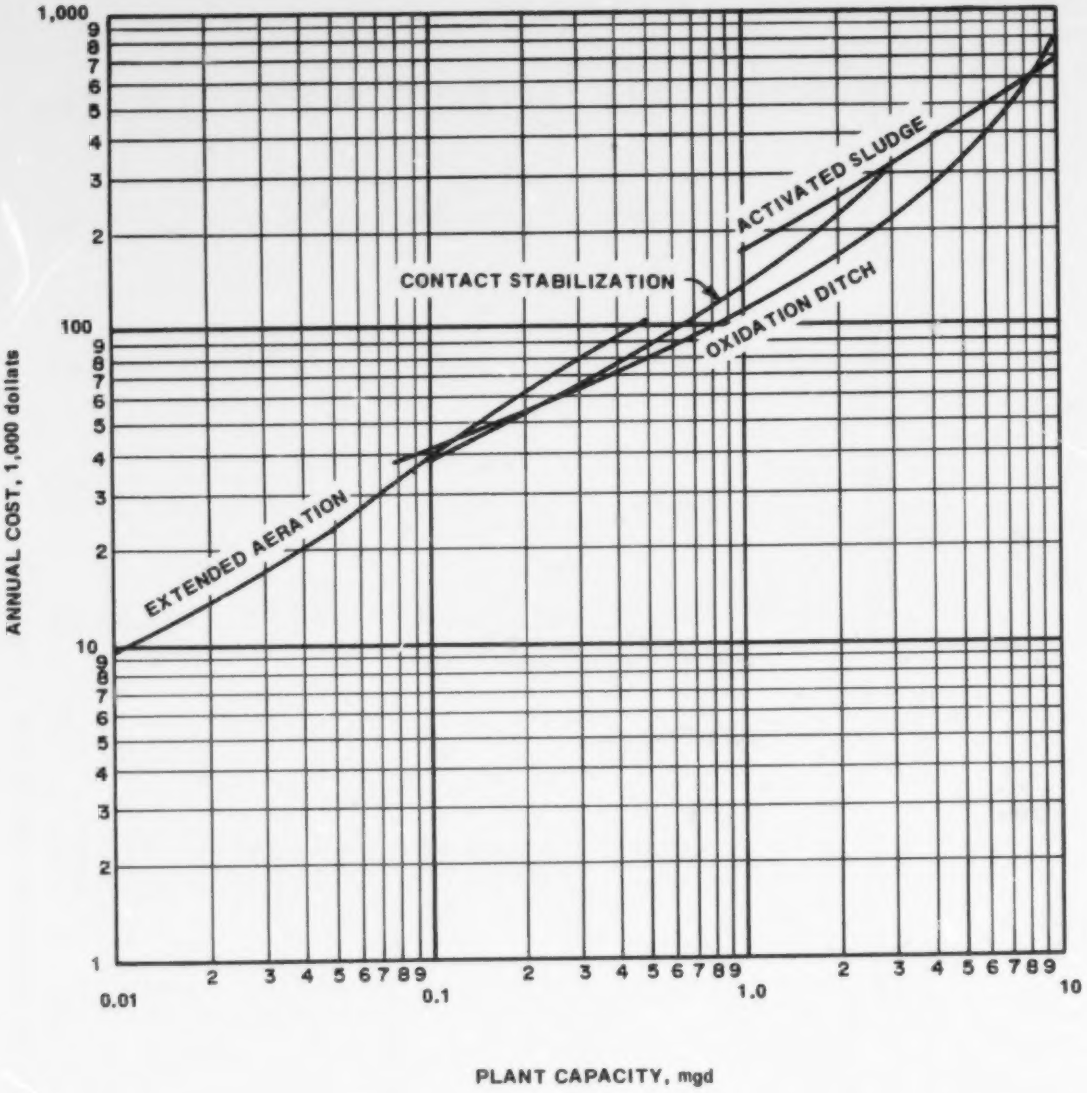
Data from Tables 15, 24, 26 and 30

Figure 36. Biological treatment process construction cost, 1976.



Data from Tables 13, 25, 27, and 31

Figure 37. Biological treatment process operation and maintenance cost, 1976.



Data from Tables 13, 15, 24, 25, 26, 27, 30 and 31
with construction costs amortised at 7% over 20 years.

Figure 38. Biological treatment process total annual cost, 1976.

TABLE 46. OPERATION PLANT OPERATION AND MAINTENANCE COSTS.

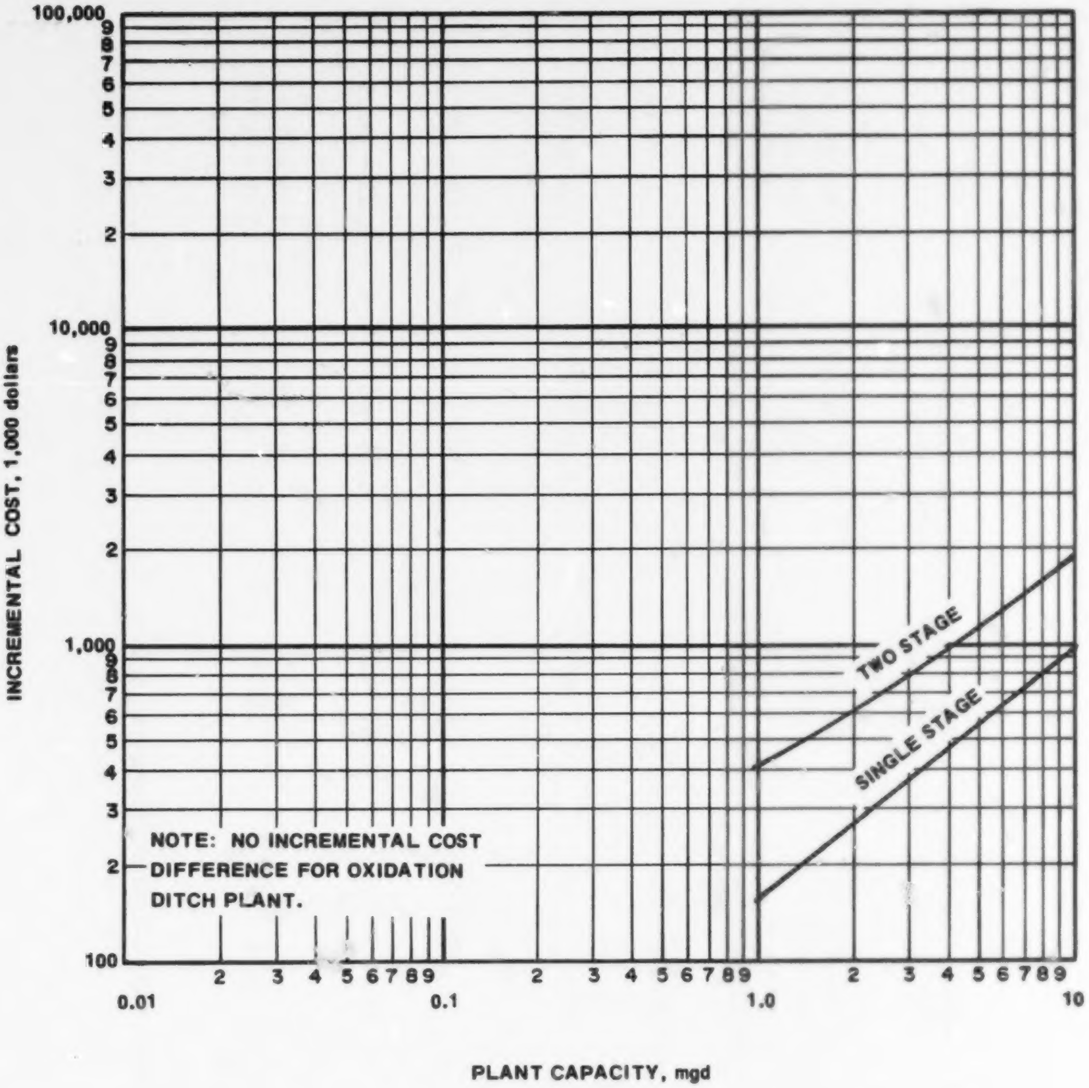
Location	Design capacity, mgd	Budget year	Staffing level, persons	Budget, $1000 Labor	Utilities	Materials	Chemicals	Total
CONVENTIONAL ACTIVATED SLUDGE								
Aspen, CO	2.0	1975	-	50	20	24	12	106
Missoula, MT	10.0	1976	-	145	53	86	17	300
Havre, MT	1.8	1976	-	65	25	15	5	110
Barstow, CA	4.5	1973	10	96	32	31	15	174
W. Sacramento, CA	2.5	1973	6	65	1	40	12	128
Carmel, CA	3.0	1973	5	73	24	54	5	156
Incline Village, NV	3.0	1973	4	44	10	44	8	106
Monterey, CA	4.6	1973	8	84	45	32	20	181
Zepher Cove, NV	3.0	1974	10	134	62	44	14	194
Snowmass, CO	0.8	1976	5	70	-	-	-	-
CONTACT STABILIZATION ACTIVATED SLUDGE								
Richwood, WV	0.5	1975	-	12	5	26	-	43
Pen Argyl, PA	0.95	1975	-	-	-	-	-	69
St. Charles, VA	0.125	1976	1.1	-	-	-	-	-
Lolo, MT	0.25	1976	2	14	11	23	1	49
Baden, PA	0.5	1976	-	47	27	15	-	89
Lancaster, PA	12.0	1976	-	97	135	114	33	379
Manteca, CA	3.2	1973	7	69	34	35	1	139
EXTENDED AERATION ACTIVATED SLUDGE								
Athens, WV	0.25	1975	-	12	5	3	1	21
Follansbee, WV	0.5	1974	3.6	18	10	5	3	36
Cheyanne, WY	0.8	1976	3	26	11	13	1	51
Elizabeth, CO	0.065	1976	1	3	1	2	1	7
Elbert, CO	0.025	1976	2	2	1	2	-	5
Columbia Falls, MT	0.5	1976	-	28	2	11	1	42
TRICKLING FILTER								
Frederick, MD	7.0	1976	-	96	22	10	63	191
Madera, CA	7.0	1973	9	60	15	7	-	82
Santa Maria, CA	6.5	1973	7	63	13	10	1	87
Chinook, MT	0.5	1976	2	12	-	8	-	20
Ft. Morgan	3.6	1976	-	30	-	40	-	70
N. Davis, UT	5.35	1976	-	54	18	27	4	103
Murray, UT	8.0	1976	-	88	14	48	24	174

Data obtained from EPA Region inspection and technical assistance reports.

TABLE 47. CHARACTERISTICS OF BIOLOGICAL TREATMENT PROCESSES
(FOR RANGE OF SIZES CONSIDERED IN THIS STUDY)

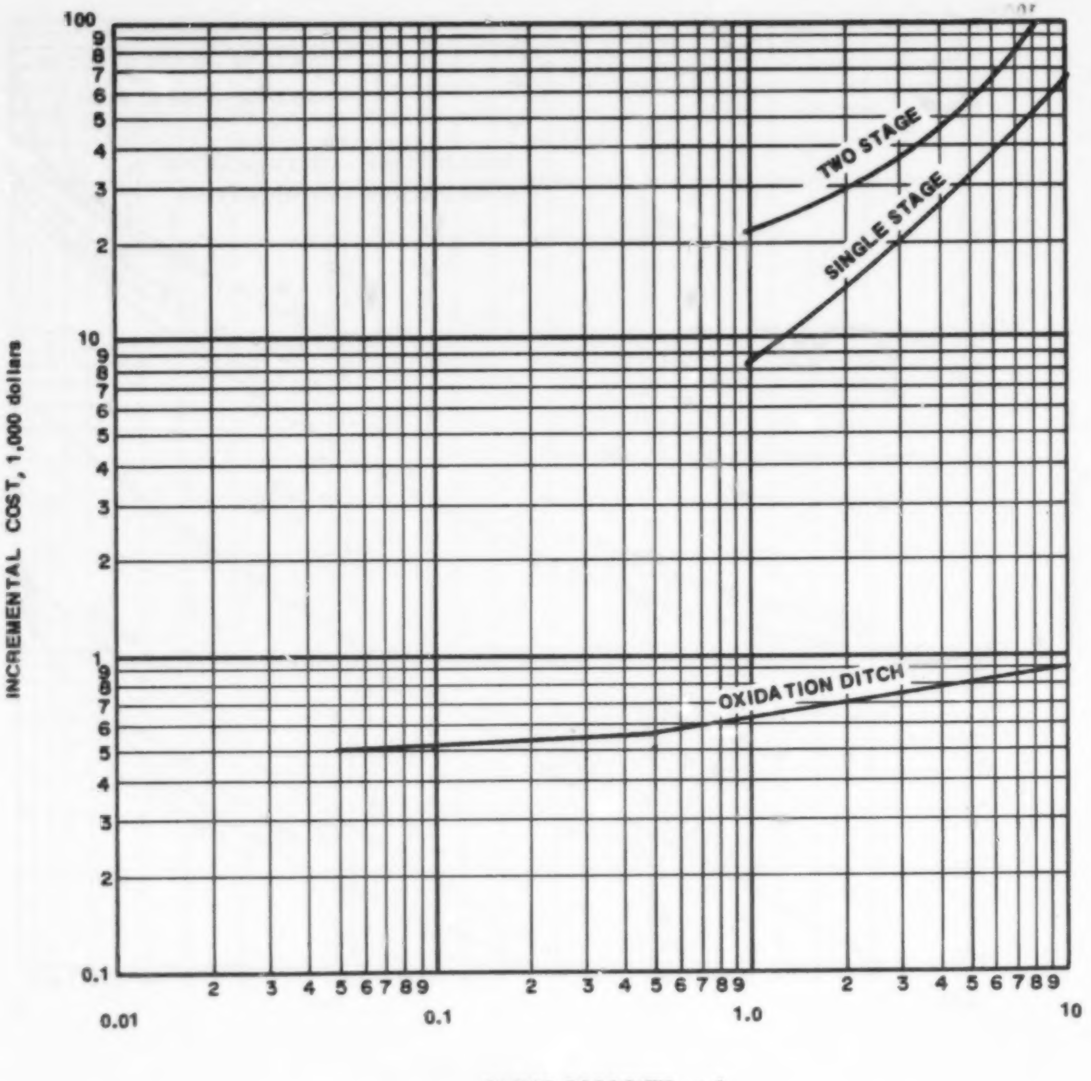
	Package extended aeration w/o primary	Package contact stab. plant w/o primary	Extended aeration oxidation ditch w/o primary	Standard activated sludge with primary
Land requirements				
Plant size range, mgd	0.01 - 0.5	0.1 to 3	0.1 to 10	1 to 10
Corresponding area requirement, acres	1	1 to 5	1.2 to 17	3.4 to 19
Construction time (Months from contract award)	9 - 12	9 - 18	12 - 18	12 - 20
Solids production, lb/lb BOD_5	0.65 - 0.75	0.65 - 0.75	0.65 - 0.75	0.65 - 0.75
Oxygen requirements (empirical) (lb oxygen/lb BOD_5 to sec. process)	1.25 1.75*	1.25 1.75*	1.25 1.75*	1.0 - 1.2 1.6 - 1.7*

*Oxygen provided to meet nitrification demands



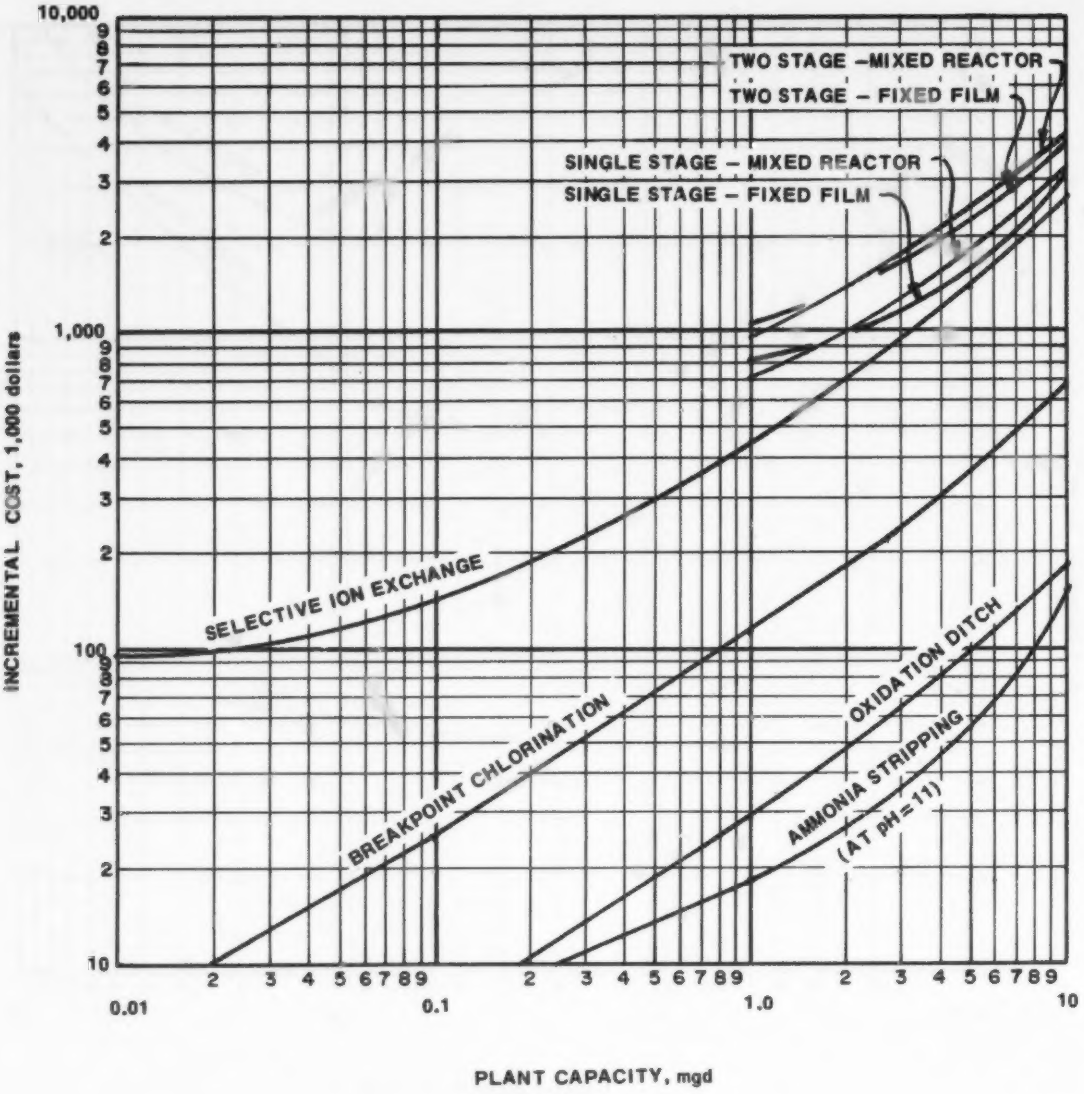
Data from Tables 34 and 36 at 20 mg/l influent NH_4–N.

Figure 39. Incremental construction cost for biological nitrification, 1976.



Data from Tables 18 and 31 at 20 mg/l influent NH_4–N.

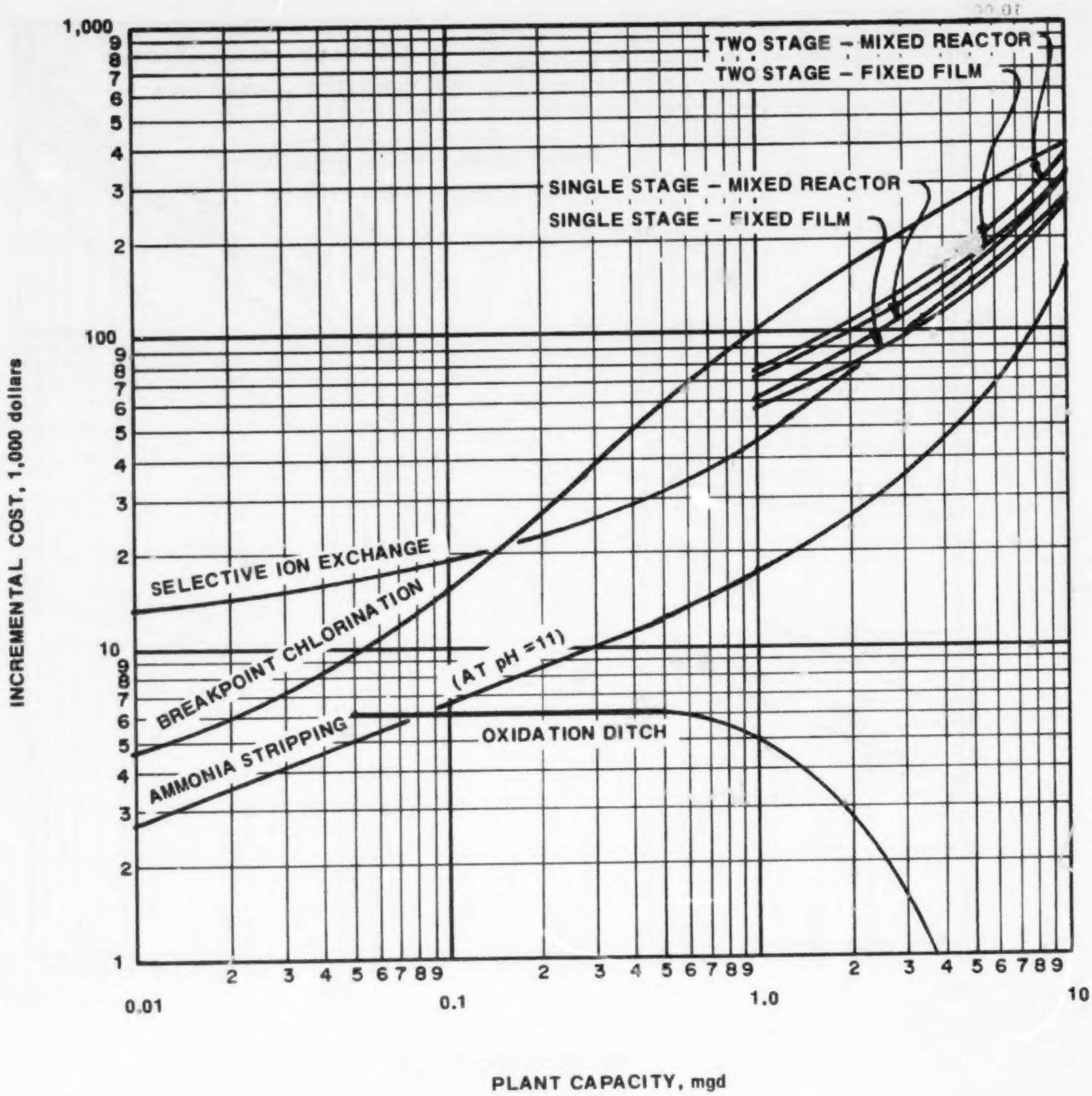
Figure 40. Incremental operation and maintenance cost for biological nitrification, 1976.



Data from Tables 15, 34, 37, 40, 42 and 44 at 20 mg/l influent NH_4–N.

Mixed reactor and fixed film denitrification include incremental costs of single or two stage biological nitrification (over costs of conventional activated sludge).

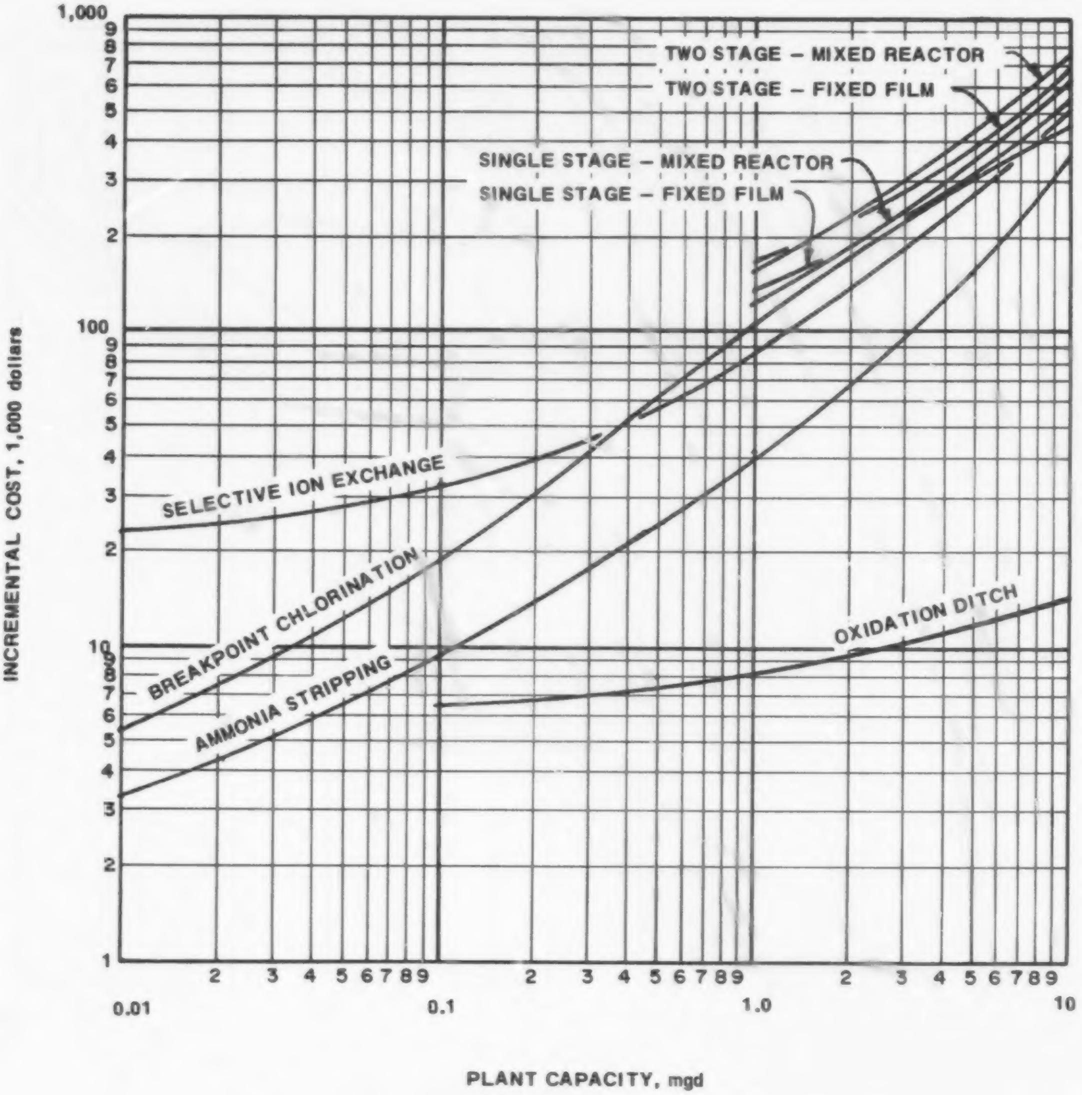
Figure 41. Incremental construction cost for biological and physical-chemical denitrification, 1976.



Data from Tables 13, 31, 41, 43 and 45 at 20 mg/l influent NH_4–N.

Mixed reactor and fixed film denitricication include incremental costs of single or two stage biological nitrification (over costs of conventional activated sludge).

Figure 42 Incremental operation and maintenance cost for biological and physical-chemical denitrification, 1976.



Data from Figures 39 and 40 at 20 mg/l influent NH_4–N.
Construction costs amortized at 7 percent over 20 years.

Mixed reactor and fixed film denitrification include incremental costs of single or two stage biological nitrification (over costs of conventional activated sludge).

Figure 43. Incremental total annual cost for biological and physical-chemical denitrification, 1976.

TABLE 48. CHARACTERISTICS OF NITROGEN REMOVAL PROCESSES

Factor	Breakpoint chlorination	Selective ion exchange	Ammonia stripping	Biological multi-stage nitrogen removal (incremental nitrif. plus denitrification)	Oxidation ditch plant (incremental over biological treatment requirements)
Temperature	No Significant Effect.	No Significant Effect.	Process performance is very temperature sensitive and freezing temperatures can cause process shutdown due to ice formation.	Process is very temperature sensitive. A decrease of reactor temperature from 20°C to 10°C requires doubling (at least of process reactor sizes - relative importance of air & water temperature depends on process selected.	Long SRT provides satisfactory operation well below 10°C & in some cases to nearly 0°C.
Influent Nitrogen Concentrations	Major impact on total costs: total costs for average of 30 mg/1 NH_4-N are 270% of costs @ 10 mg/l	Relatively minor impact on total costs: total costs for average of 30 mg/l NH_4-N are 125% of costs @ 10 mg/l	Process provides essentially fixed % of removal over a wide range of ammonia concentrations.	Construction & O & M costs generally increase with increased influent ammonia concentration.	Influent ammonia concentration has little effect up to 50 or 60 mg/l.
Land Requirements	5-10 sq ft/mgd	200-300 sq ft/mgd.	350-700 sq ft/mgd	13,000 to 26,000 sq ft/mgd	Incremental land requirements over biological treatment (70,000-500,000 sq ft/mgd) are nil.
Operational Considerations	Simple & efficient, but requires high chlorine dosages which must be evaluated as related to water quality.	Efficient, dependable, & controlable for nitrogen removal, but operation & equipment are relatively complex.	Simple operation but subject to poor operation in cold weather, shutdown in freezing weather, & scaling problems. Process requires significant pH adjustment prior to and after stripping.	Subject to biological upsets which may disrupt process for many days.	Same as biological plus denitrification efficiency, reliability, & operability have not been demonstrated widely.
Relative Energy Consumption	Low, typically 35 kwh/mg	Medium, typically 250 kwh/mg	Highest, typically 1500 kwh/mg	High, typically 900 kwh/mg	Energy consumption is less than biological requirements.

SECTION 8

DISCUSSION AND EVALUATION

PROCESS AND DESIGN

There are many possible oxidation ditch plant variations related to channel configuration, detention time, operation mode, aeration equipment, sludge handling, and similar considerations. Many of these variations are minor and have little impact on plant operation and performance.

Current practice in the United States is to design oxidation ditch plants with either a single or multiple concentric aeration channel. Primary sedimentation is seldom provided; however, the raw sewage is normally screened or comminuted and then degritted. The oxidation ditch is generally designed for 8 to 24 hours aeration detention time with 24-hour detention being most common. This results in a loading of approximately 12 to 13 lb BOD_5 per 1,000 cu ft of aeration volume for normal domestic sewage assuming 0.17 lb BOD_5 and 100 gal per capita per day with 24-hour aeration detention. The sludge age is typically 10 to 33 days.

The conservative biological design of oxidation ditch plants provides flexibility in operation, a stable sludge, and performance which is above the average of all other competing secondary processes.

PROCESS EQUIPMENT

Several manufacturers supply aeration equipment for oxidation ditch plants. Generally, this equipment consists of horizontal brush, cage, or disc type aerators specifically designed for oxidation ditch plants, but useful to other aeration applications such as lagoons. The Carrousel plant, licensed by Envirotech, uses vertical aeration equipment.

The aerator manufacturers provide application information for their equipment. Even though a number of the aerators are very similar in appearance, the manufacturers application information indicates a wide variation in oxygen transfer rate. Some of this variation may be due to aerator design, but most of it is probably due to differences in aerator test conditions. Typically, aerator oxygen transfer efficiency is in the range of 3 to 5 lb O_2 per hp-hr-lineal foot of aerator length according to manufacturers published literature. Actual testing of aerators was beyond the scope of this study, but information from actual installations would indicate that manufacturers aerator recommendations provide adequate oxygen in nearly all applications. A number of the plants were underloaded organically so the observations made do not positively indicate the adequacy of aeration at design load, but the trend was clear.

Most of the other process equipment used in oxidation ditch plants is, or can be, standard industry equipment as available from a number of manufacturers. Therefore, oxidation ditch plant equipment can be specified and bid competitively as is common practice with municipal wastewater treatment plants.

The Carrousel plant is patented and must be designed and operated under license, however, the process equipment for the plant is not covered under the patent and may be obtained from a number of manufacturers. The other oxidation ditch plant configurations are not patented and may be used without license fee.

PERFORMANCE

Analysis was made of extensive data from operating oxidation ditch plants. The average performance of the plants studied was equal to or better than a 20-20 effluent 85 to 90 percent of the time and a 30-30 effluent 95 percent of the time. The best of the plants studied met a 10-10 effluent 99 percent of the time. The reliability curves indicate slightly better performance in summer than winter, but the difference is small. Of the oxidation ditch plants analyzed, the worst met a 20-20 effluent 50 percent of the time and a 30-30 effluent 70 to 80 percent of the time with the effluent seldom exceeding 60 mg/l BOD_5 and TSS.

Average removal of BOD_5 is 93 percent and TSS is 94 percent with a range of about 85 to 98 percent for each parameter among all oxidation ditch plant data analyzed.

There is little difference in BOD_5 and TSS performance, on the average, between plants of various sizes.

The performance of oxidation ditch plants was compared to various competing processes such as standard activated sludge, contact stabilization, extended aeration, rotating biological contactor, and trickling filter. The average BOD_5 and TSS performance of oxidation ditch plants exceeded the corresponding performance of the competing processes; in some cases by a wide margin. A summary of the comparative performance is shown in Table 23.

CONSTRUCTION

Construction costs for oxidation ditch plants were determined from actual plant experiences and compared to construction costs for other biological treatment processes. A comparison of the costs is shown in

Figure 36. Generally, the extended aeration package plant is the lowest cost for small sizes, the contact stabilization plants are lowest cost in the mid range of sizes, and the oxidation ditch is lowest cost for larger sized plants. The costs of alternative plants are close enough in the range of 0.1 to 10 mgd, however, that local factors and the exact process design will have a significant effect on the relative construction costs.

The effect of ditch configuration on construction costs was also investigated for plants up to 1 mgd. This comparison is shown in Table 18. For small plants the shallow sloped side ditch is most cost effective and the deep ditch becomes more cost effective as the capacity increases. For plants less than one mgd in size, the effect of configuration is minor, but for large plants over one mgd, the deep configuration is being used frequently.

Space requirements for oxidation ditch plants are shown in Table 19 based on a reasonable layout of the required facilities including sludge drying beds. These requirements are very comparable to similar requirements for other treatment processes as shown in Table 47.

OPERATION AND MAINTENANCE

The operation and maintenance requirements for oxidation ditch plants were determined and compared to the corresponding requirements for competing processes. The operation and maintenance requirements are summarized in Table 13. These requirements are based on actual plant experiences to the extent possible. The actual costs for labor and electrical energy are shown to illustrate the extent of variation of unit costs. Labor unit costs varied from about $2.00 to $10.00 per hour and were generally lower for smaller plants. For purposes of comparison, a uniform unit cost of $9.00 per hour was used for labor and $0.03 per kwh was used for electrical energy. A comparison of operation and maintenance total cost for various biological treatment processes is shown in Figure 37. The following comments relate to operation and maintenance costs only. Package extended aeration plants are the lowest cost for small capacity applications below about 0.1 mgd. This is probably due to the compact plant configuration and the minimal amount of mechanical equipment. The standard activated sludge process is the lowest cost for larger capacity plants generally above 2 or 3 mgd although the difference between standard activated sludge and oxidation ditch costs is small up to 10 mgd. The major variable relates to the staffing practices of particular plants because at some point within the capacity range of 1 to 10 mgd full time, 24-hour staffing is generally desireable.

Within the capacity range of 0.1 to 2 or 3 mgd the oxidation ditch plant is very competitive in operation and maintenance cost. This is due to the conservative design, process simplicity, and relatively easy sludge handling associated with the oxidation ditch plant.

An analysis of total annual costs is shown in Figure 38 with construction costs amortized at 7 percent over 20 years. The oxidation ditch plant is cost effective over the range of 0.1 to almost 10 mgd although the differences are small at some points. Therefore, it is important to compare the major alternatives in each specific case and consider the local factors. Package extended aeration plants are most cost effective for the small capacity plants below 0.1 mgd.

As a group, oxidation ditch plants are simple to operate and reliable. The process simplicity, conservative design, and use of standard equipment contributes to the very satisfactory operating experience. These plants perform well for extended periods with little or no operator attention. With few exceptions, operators and administrators are pleased with their plants.

Oxidation ditch plants are subject to periodic problems, primarily mechanical. These problems are detailed in Section 5. The major problems are as follows:

1. Problems can be expected every 2 to 5 years with each aerator unit. These problems include bearings, seals, flexible couplings, gear reducers, and loss of teeth on brush aerators. Some failures of the aerator torque tube have been experienced, but this is rare.

2. Comminutors require regular maintenance and cleaning for satisfactory operation.

3. Spray from the aerators is a problem when it blows onto walkways and other access areas. It results in slippery slime accumulations or ice; both are hazards to personnel. In addition, this spray blows onto bearings and drive units contributing to early failure.

4. There are a number of other reported minor problems with return sludge pumping, cleaning of weirs and scum boxes, clarifiers, and similar items common to other types of plants.

Generally, the operational experience with oxidation ditch plants has been very good and careful design and operation will overcome a number of the problems mentioned.

Manufacturers are helping to provide solutions to some of the problems by providing better seals, double sealed bearings, self-aligning bearings, better designs for aerator support structure, and shields or "dry" compartments for bearings and drive units. These considerations should contribute to even better performance in the future.

Cold weather affects the operation and maintenance of oxidation ditch plants depending on the severity of the cold. Areas where freezing is only intermittent and of short duration require very little special consideration. In moderately cold areas where spray from aerators will freeze and accumulate, the aerators are generally fitted with covers which may be heated. A major consideration is the accumulation of chunks of ice in the ditch which might damage the aerator blades or brushes as the chunks pass through. A baffle at water level upstream of each aerator may be helpful to keep ice chunks out of the aerator. Severely cold areas may require special design considerations for satisfactory operation and maintenance, but this varies from case to case. An outdoor oxidation ditch plant is operating quite satisfactorily in Fairbanks, Alaska with minimum protective features; however, in other very cold areas special protective features have been used such as covering of ditches and final clarifiers and housing of equipment requiring regular maintenance. Oxidation ditch plants generally require cold weather protective considerations similar to other processes for a given climate.

SLUDGE HANDLING

Oxidation ditch plants are operated similar to other activated sludge plants with the settled activated sludge returned from the final clarifier to the oxidation ditch. The considerations are essentially the same as for the standard activated sludge process. The sludge should be returned continuously at a rate generally about 30 to 50 percent of the plant average daily flow. The sludge return rate should be adjustable and measureable. The return sludge flow rate is generally not varied in proportion to the plant flow rate, but can be if desired.

Normally, a portion of the return sludge is wasted to maintain the plant solids balance. Some plants do not waste sludge at all, but this practice is not recommended because of the buildup of inert solids and the eventual uncontrolled wasting (plant upset) which almost invariably results from this type of operation.

It is recommended that sludge be wasted on a regular schedule to maintain the desired MLSS in the ditch.

Typically, waste sludge is handled in one of two ways at oxidation ditch plants. The most common is to utilize extended aeration (24-hour aeration detention) and to waste sludge directly to sand drying beds. The second is to provide less aeration detention (8 to 15 hours) and waste sludge to an aerobic digester then to sand drying beds. In some cases the liquid waste sludge is spread on land without drying. There are a few other variations, but most plants waste sludge directly to sand drying beds.

At nearly all oxidation ditch plants visited, the waste sludge was dewatered in open drying beds without any further treatment. Without exception, plant personnel indicated an absence of odor and an absence of nuisance complaints. No odors were detected during site visits to approximately 20 plants of various sizes, locations, and types.

A number of plants reported a shortage of sand drying bed capacity and slow dewatering of the sludge on sand beds. This would indicate the need for adequate design for the particular region. It is desireable to have adequate drying capacity so that wasting can be scheduled according to process needs rather than by available drying bed capacity. It is possible to store sludge in the oxidation ditch by allowing the MLSS to increase, but this must be carefully monitored to prevent plant upsets. It is far better to waste sludge at regular intervals.

If sludge is wasted directly to sand beds it is important that the plant be properly operated so that a stable sludge is produced. If the sludge is not well stabilized odors may be generated during the drying process.

Some regulatory agencies may require additional sludge stabilization prior to disposal and these requirements should be taken into consideration in plant designs. Sludge storage may be required in some applications; for instance, if sand drying beds are provided, but are not usable during some periods of the year.

It is important in the design of any solids handling process to assess the impact of the non-biodegradable portion of the plant solids. The non-biodegradable solids will accumulate in the aeration process and will not be further reduced by extended aeration or aerobic digestion and eventually must be wasted and disposed of. The non-biodegradable fraction of the influent solids will vary widely, but may typically be 30 to 50 percent of the raw sewage solids. This entire fraction of non-biodegradable solids enters the aeration process when the plant has no primary settling.

NITRIFICATION AND NITROGEN REMOVAL

A standard oxidation ditch plant is capable of essentially complete nitrification when designed for 24-hour aeration detention treating normal domestic sewage. Generally, the only modifications required are operational in nature provided adequate aeration capacity is available to maintain a positive dissolved oxygen level in the ditch, normal return sludge capability is provided, and an adequate final clarifier(s) is provided. If the MLSS is maintained in the range of 2,600 to 5,000 mg/l the plant should achieve complete nitrification down to mixed liquor temperatures near freezing. In some cases, the mixed liquor pH may vary enough to require chemical adjustment to the optimum range of 7.5 to 9.5, but these cases are rare. There are a number of design and operational parameters which effect operation for optimal nitrification as outlined in Section 4, but for the typical plant, normal operation will produce a very high degree of nitrification.

Oxidation ditch plants are capable of substantial nitrogen removal, however, the design and operation is rather critical if consistently high levels of nitrogen removal are to be achieved. It has been demonstrated that 40 to 80 percent nitrogen removal can be achieved and under favorable conditions 70 to 80 percent removal should be possible with mixed liquor temperatures above 15°C.

Oxidation ditch plants are able to achieve single stage nitrogen removal because alternating aerobic and anoxic zones can be maintained within the same ditch. This requires careful control of the aeration. With the deep ditch type plants it is also possible to produce aerobic and anoxic zones due to vertical stratification. This stratification was shown at the Vienna-Blumenthal plant(19). Work by Drews, et al(17,18) also showed that multiple concentric channel type plants can be operated for single stage nitrogen removal.

Normally, an auxiliary source of carbon is required for the denitrification step and this can be provided by introducing the raw sewage into the ditch near the transition between the aerobic and anoxic zones along with the return sludge.

Some positive means must be provided for easy control of the aeration so that the aerobic-anoxic zones can be maintained under varying plant flow and loading. This can be accomplished by varying aerator immersion depth, aerator speed, number of aerators in operation, or operating the aerators on an on-off cycle. Floating aerators may have some advantage because their position in the ditch can be changed. Under some cases of low oxygen demand the aerators may not impart sufficient velocity to maintain the solids in suspension in the ditch. In this case, an auxiliary velocity device may be necessary to impart additional velocity without adding oxygen such as a submerged propeller.

Ideally, the use of an automatic dissolved oxygen control system would assure maintenance of the required aerobic and anoxic zones with minimum operator attention, but this has not been demonstrated.

Generally, the operational considerations for nitrogen removal are about the same as for nitrification except for the aerobic-anoxic zones.

The oxidation ditch operation and maintenance costs for nitrification and nitrogen removal are shown in Table 13. The incremental oxidation ditch operation and maintenance costs for nitrification are very small. The operation and maintenance costs for nitrogen removal are actually less than for normal oxidation ditch plant operation in larger plant sizes because of savings in electrical energy due to controlled aeration. In essence, the incremental operation and maintenance costs for nitrification or nitrogen removal over those for normal oxidation ditch plant operation are nil.

Construction costs for nitrification are the same as for a normal oxidation ditch plant and for nitrogen removal are about 4 to 6 percent higher.

A comparison of the oxidation ditch to single and two-stage biological nitrification is shown in Figures 39 and 40. Each process will provide essentially complete nitrification, however, the oxidation ditch plant requires no additional construction cost and very little additional operation and maintenance cost over the basic oxidation ditch treatment plant.

A comparison of various nitrogen removal alternatives is shown in Figures 41, 42, and 43. Ammonia stripping and breakpoint chlorination are the only close competitors (in small plants) to the oxidation ditch plant, but both have serious process limitations. These limitations are outlined in Table 48. The incremental costs for ammonia stripping do not include raising the pH prior to or lowering the pH after stripping. If very high nitrogen removal is required, the oxidation ditch plant would not be suitable, but for removals up to 80 percent the oxidation ditch plant is cost effective compared to the other alternatives.

EFFECTIVE APPLICATION

The effective application of oxidation ditch plants requires more than consideration of the aeration process itself. Many other factors contribute to overall satisfactory performance.

The raw sewage should be at least coarse screened or comminuted before entering the ditch to prevent plugging of return sludge valves and pumps. The screen should have openings no wider than 3/4 to 1 inch. A number of plants have no grit removal facilities and have been in operation for several years without any adverse effects from grit accumulation. Therefore, removal of grit may be desireable, but lack of grit removal does not seem to cause serious operating or maintenance problems. It is likely that grit may have to be removed from these ditches after some period of operation.

The final clarifier design and sizing is very important to overall plant performance especially if the discharge requirements are stringent. It is recommended that final clarifiers be designed for an overflow rate of 400 to 500 gpd/sq ft at average plant flow and 1,000 to 1,200 gpd/sq ft at peak flows. Clarifiers should be designed for a solids loading of 30 lb/day/sq ft. Sidewall depths should preferably be 12 to 14 feet. A minimum of two final clarifiers should be considered for all plants. Deep scum baffles and double skimmer arms are helpful if rising solids are expected because of denitrification in the final clarifier. Return sludge should be withdrawn from the final clarifier continuously and the rate measured. This sludge can be returned to the ditch or to the raw sewage lift station.

Provisions should be made for easy sludge wasting and some means should be available for measuring the quantity of sludge wasted.

Drying beds are the most common method of dewatering, but the beds should be adequately sized for the quantity of sludge to be wasted (considering the non-biodegradable solids) and the climate. Additional sludge storage may be necessary or desireable in some applications to assure that sludge can be wasted and handled under all operating conditions.

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LIST OF METRIC CONVERSIONS

English Unit	Multiplier	Metric Unit
acre	0.405	ha
cfm	0.028	cu m/min
cfs	1.7	cu m/min
cfs/acre	4.2	cu m/min/ha
cfs/sq mile	0.657	cu m/min/sq km
cu ft	0.028	cu m
cu ft	28.32	l
cu in.	16.39	cu cm
cu yd	0.765	cu m
cu yd/mile	0.475	cu m/km
cu yd/sq mile	0.29	cu m/sq km
°F	0.555(°F-32)	°C
ft	0.3048	m
gal	0.003785	cu m
gal	3.785	l
gpd/acre	0.00935	cu m/day/ha
gpd/cu yd	5.0	l/day/cu m
gpd/ft	0.0124	cu m/day/m
gpd/sq ft	0.0408	cu m/day sq m
gpm	0.0631	l/sec
gpm/sq ft	40.7	l/min/sq m
hp	0.7457	kw
in.	2.54	cm
lb	0.454	kg
lb/1,000 cu ft	16.0	g/cu m
lb/day/sq ft	4,880	g/day/sq m
lb/ft	1.51	km
lb/mil gal	0.12	g/cu m
mgd	3,785	cu m/day
mile	1.61	km
pcf	16.02	kg/cu m
psf	4.88	kg/sq m
psi	0.0703	kg/sq cm
sq ft	0.0929	sq m
sq in.	6.452	sq cm